Contents

Chapter 7 Substructure Design

7.1 General Substructure Considerations

Note that in the following guidelines where reference is made to AASHTO, the item can be found in the current AASHTO LRFD Bridge Design Specifications, with Interims.

7.1.1 Foundation Design Process

A flowchart is provided in Figure 7-1 which illustrates the overall design process needed to accomplish an LRFD foundation design. The Bridge Office)BO_ and the Geotechnical Branch (GB) have been abbreviated. The steps in the flowchart are defined as follows:

Conceptual Bridge Foundation Design

This design step results in an informal communication produced by the Geotechnical Branch at the request of the Bridge and Structures Office which provides a brief description of the following.

- Anticipated soil site conditions
- Maximum embankment slopes
- Foundation types and geotechnical hazards such as liquefaction

In general, these recommendations rely on existing site data. Site borings may not be available and test holes are drilled later. The geotechnical recommendations provide enough information to select a type of foundation for an initial Bridge Preliminary Plan.

Develop Site Data and Preliminary Bridge Plan

In the second phase, the Bridge and Structures Office obtains site data from the region (see WSDOT *Design Manual*) and develops the Preliminary Bridge Plan. The preliminary pier locations determine soil boring locations at this time. The Geotechnical Division will also require the following information to continue the preliminary geotechnical design.

- Structure type and magnitude of settlement the structure can tolerate (both total and differential).
- At abutments – Approximate maximum top of foundation elevation.
- At interior piers – The number of columns; Whether a single foundation element supports each column or one foundation element supports multiple columns.
- At stream crossings – Pier scour depth, if known. Typically, the Geotechnical Division will pursue this issue with the OSC Hydraulics Office.
- Any known structural constraints that affect the foundations' type, size, or location.
- Any known constraints that affect the soil resistance (utilities, construction staging, excavation, shoring and falsework).

Preliminary Foundation Design

The third phase is a request by the Bridge and Structures Office for a preliminary foundation memorandum. The Geotechnical Division memo will provide preliminary soil data required for structural analysis and modeling. This includes any subsurface conditions and the preliminary subsurface profile.

The concurrent geotechnical work at this stage includes:

- Completion of detailed boring logs and laboratory test data
- Development of foundation type, soil capacity, and foundation depth
- Development of static/seismic soil properties and ground acceleration
- Recommendations for constructability issues

Structural Analysis and Modeling

In the fourth phase, the Bridge and Structures Office performs a structural analysis of the superstructure and substructure using a bridge model and preliminary soil parameters. Through this modeling, the Bridge Engineer determines loads and sizes for the foundation based on the controlling LRFD limit states. Structural and geotechnical design continues to investigate constructability and construction staging issues during this phase.

In order to produce a Final Geotechnical Report, the Bridge and Structures Office provides the following structural feedback to the Geotechnical Engineer.

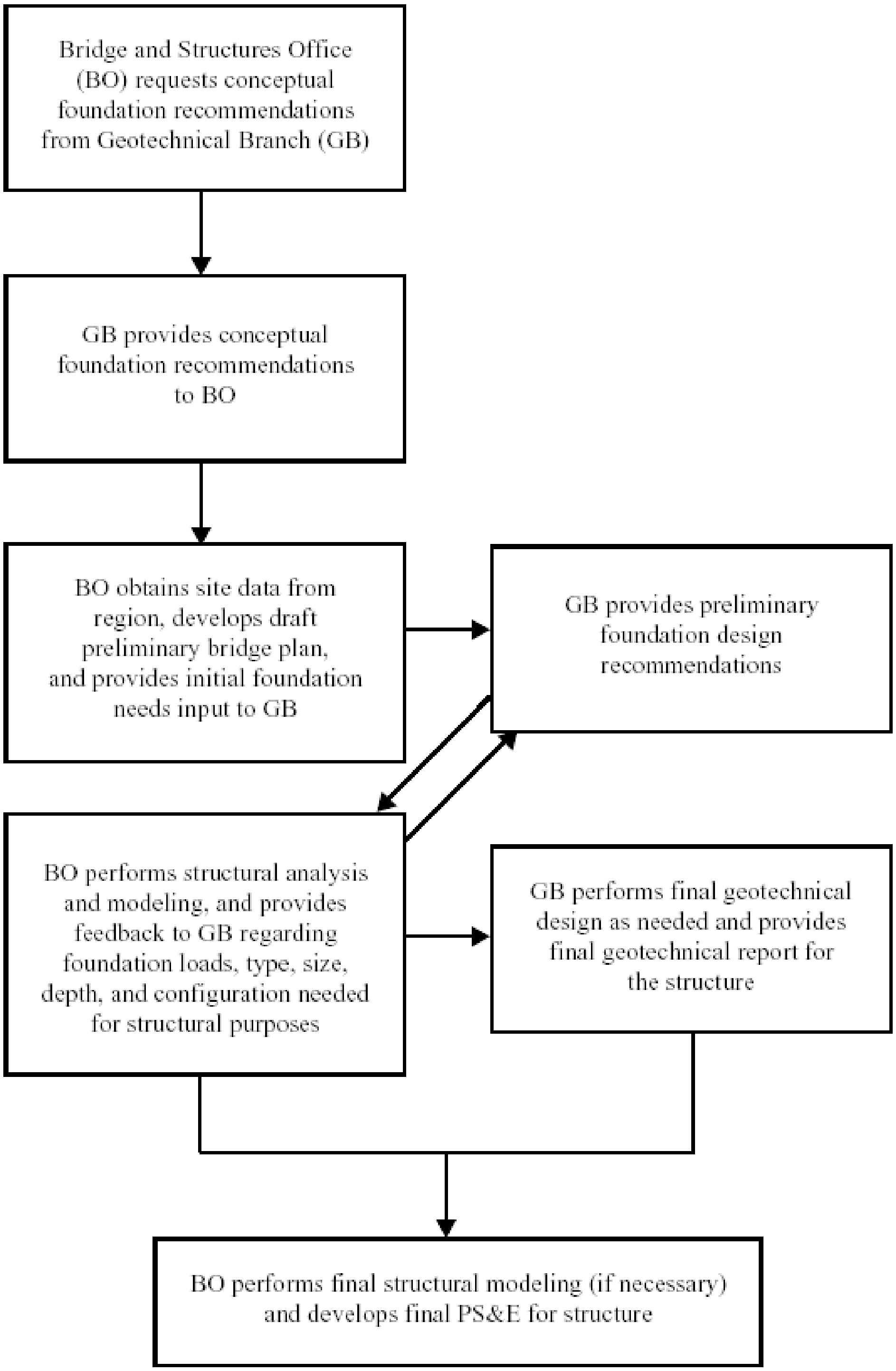
- Foundation loads for service limit state and strength limit state.
- Foundation size/diameter and depth required to meet structural design.
- Foundation details that could affect the geotechnical design of the foundations.
- Size and configuration for deep foundation groups

Final Foundation Design

The last phase completes the geotechnical report and allows the final structural design to begin. The preliminary geotechnical assumptions are checked and recommendations are modified, if necessary. The final report is complete to a PS&E format since the Project Contract will contain referenced information in the Geotechnical Report, such as:

- All geotechnical data obtained at the site (boring logs, subsurface profiles, and laboratory test data)
- All final foundation recommendations
- Final constructability and staging recommendations

The Bridge Engineer reviews the final report for new information and confirms the preliminary assumptions. With the foundation design complete, the final bridge structural design and detailing process continues to prepare the Bridge Plans.



Overall Design Process for LRFD Foundation Design
Figure 7-1

7.1.2 Foundation Limit States and Factors

The controlling limit states for WSDOT projects for Substructure Design are described as follows:

Strength I	Relating to the normal vehicular use
Strength III	Relating to the bridge exposed to wind
Strength IV	Relating to temperature fluctuations, creep, and shrinkage
Strength V	Relating to the normal vehicular use and wind
Extreme-Event I	Relating to earthquake
Service I	Relating to normal operational use and wind

The basic equation for load and resistance factor design (LRFD) states that the loads multiplied by factors to account for uncertainty, ductility, importance, and redundancy must be less than or equal to the available resistance multiplied by factors to account for variability and uncertainty per the AASHTO LRFD specifications. WSDOT current practice sets η_i equal to 1.0 for structural design. The basic equation, therefore, is as follows:

$$\Sigma\eta_i\gamma_i Q_i \leq \phi R_n$$

η_i = Factor for ductility, redundancy, and importance of structure

γ_i = Load factor

Q_i = Load (i.e., dead load, live load, seismic load, etc.)

ϕ = Resistance factor

R_n = Nominal or ultimate resistance

Substructure Load Factors

The load combinations, limit states and load factors (γ_i) used for foundation design are in accordance with AASHTO, Table 3.4.1-1. Design loads are factored after distribution to the foundation through structural analysis or modeling.

Transient Load Factors -- Live Load (γ_{EQ})

The Extreme Event-I transient load factor for live load, γ_{EQ} as specified in the LRFD Table 3.4.1-1, may vary from 0.0 up to 0.5. For WSDOT bridge designs, the live load factor, γ_{EQ} for the majority of bridges, shall be equal to 0.0. For bridges located on routes in urban areas susceptible to daily, heavy congestion, γ_{EQ} shall be equal to 0.5 unless otherwise directed by the WSDOT Bridge Design Engineer. The γ_{EQ} factor should apply to the live load force effect obtained from the bridge live load elastic analysis. Live load mass will be ignored in the dynamic analysis.

Permanent Load Factors - DD

The load factors for permanent loads (γ_p) will be in accordance with Table 3.4.1-2 of the AASHTO LRFD specifications, with the exception of downdrag (DD). Downdrag load factor (γ_{DD}) will be specified in the Geotechnical Report for the Bridge.

Response Modification Factors (R)

The Response Modification Factor for all bridges is based on the Importance Category of "Other" as specified in LRFD Table 3.10.7.1-1, unless otherwise directed by the WSDOT Bridge Design Engineer. The R-factors for column design apply only to the elastically computed moments from the seismic analysis. The R-factor for column shear design will always be 1.0.

AASHTO indicates Importance Categories of Critical, Essential and Others for The Response Modification Factor (R). Bridge columns designed and detailed in accordance with the criteria for "Other" bridges exhibit good seismic performance and it may be unnecessary and uneconomical to design bridges for Critical or Essential categories. However, if the WSDOT Bridge Design Engineer identifies a bridge as Essential or Critical, the corresponding R-factor should be used for substructure design.

A. Substructure Permanent Load Factors

Table 7-2 provides general guidelines for when to use the maximum or minimum shaft/pile/column strength limit state permanent load factors for axial capacity, uplift, and lateral loading.

In general, substructure design should use unfactored loads to obtain force distribution in the structure, and then factor the resulting moment and shear for final structural design. All forces and load factors are as defined previously.

Axial Capacity	**Uplift**	**Lateral Loading**
DC_{max}, DW_{max}	DC_{min}, DW_{min}	DC_{max}, DW_{max}
DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} causing shear
DC_{min}, DW_{min} for shear	DC_{min}, DW_{min} for resisting shear	DC_{min}, DW_{min} resisting resisting shear
DC_{max}, DW_{max} for causing moments	Use DC_{max}, DW_{max} for causing moments	Use DC_{max}, Dw_{max} for causing moments
DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments
EV_{max}	EV_{min}	EV_{max}
DD = Varies	DD = Varies	DD = Varies
EH_{max}	EH_{max} if causes uplift	EH_{max}

Maximum or Minimum Substructure Load Factors for Strength Limit State permanent Loads (γ_p)
Table 7-2

7.1.3 Substructure and Foundation Loads

Figure 7-3 below provides a common basis of understanding for load location and orientations for substructure design. This figure also shows elevations required for abutment and substructure design. Note that for shaft and some pile foundation designs, the shaft or pile may form the column as well as the foundation element.

Spread footings usually have a design orientation normal to the footing. Since bridge loads are longitudinal and transverse, skewed superstructure loads are converted (using vector components) to normal and parallel footing loads. Deep foundation analysis usually has a normal/parallel orientation to the pier in order to simplify group effects.

Substructure Directional Forces

Figure 7-3

7.1.3 Substructure and Foundation Loads (continued)

Substructure elements are to carry all of the loads specified in AASHTO. Selecting the controlling load conditions requires good judgment to minimize design time.

Bridge design will consider construction loads to ensure structural stability and prevent members from overstress. For example, temporary construction loads can overload a single column pier by placing all of the precast girders on one side. The plans will show a construction sequence and/or notes to avoid unacceptable loadings.

On curved bridges, the substructure design will consider the eccentricity resulting from the difference in girder lengths. When superstructure design uses a curved girder theory, such as the V-Load Method, the reactions from such analysis must be included in the loads applied to the substructure.

Dead Loads - DC

Substructure design will account for all anticipated dead load conditions. Sidesway effect shall be included where it tends to increase stresses.

Live Loads - LL

The dynamic allowance (IM) will be applied in accordance with AASHTO 3.6.2 and is not included in the design of buried elements of the substructure. Portions of the abutments in contact with the soil are considered buried elements.

Lane reduction factors as described in AASHTO "Reduction in Load Intensity" are applied to the number of lanes for each load case.

The HL93 loading is distributed to the substructure by placing wheel line reactions in a lane configuration that generates the maximum stress in the substructure. A wheel line reaction is $^{1}/_{2}$ of the HL93 reaction. Live loads are considered to act directly on the substructure without further distribution through the superstructure, see Figure 7-4. Normally, substructure design will not consider live load torsional or lateral distribution nor any live load sidesway effects. GTSTRUDL will include live load sidesway.

Live Load Wheel Line Destribution to Substructre
Figure 7-4

For steel and prestressed concrete superstructures where the live load is transferred to substructure through bearings, cross frames or diaphragms, the girder reaction may be used for substructure design.

Live load placement is dependant on the member under design. Some examples of live load placement are as follows.

The exterior vehicle wheel is placed 2 feet from the curb for maximum crossbeam cantilever moment or maximum eccentric foundation moment.

For crossbeam design between supports, the HL93 lanes are placed to obtain the maximum moment in the member; then re-located to obtain the maximum shear or negative moment in the member.

For column design, the design lanes are placed to obtain the maximum transverse moment at the top of the column; then re-located to obtain the maximum axial force of the column.

Down Drag Force - DD

The Geotechnical Report will provide the downdrag force. The down drag force (DD) is a load applied to the pile/shaft with the load factor (γ_{DD}) specified in the Geotechnical Report. Generally, transient loads (LL) are less than the downdrag force and should be omitted when considering DD forces. In other words, the transient loads reduce downdrag forces and are ignored for the structural design. The WSDOT GDM Section 8.6.2 provides a more in-depth discussion of Down Drag.

Earthquake Loads - EQ

Earthquake loads on elements of the substructure are described in AASHTO 3.10, Earthquake Effects. The design acceleration coefficient and site coefficient will be given in the Geotechnical Report. The seismic analysis requirements are stated in AASHTO Section 4.7. The resulting loads shall be taken in any horizontal direction to give maximum design load for the substructure element.

The intermediate pier(s) of each unit of a multispan continuous structure shall be designed to resist the entire longitudinal earthquake force for that unit (unless the end piers are an integral part of the superstructure). The calculated longitudinal movement shall be used to determine the shear force developed by the pads at the abutments. The neoprene modulus of elasticity at 70°F (21°C) shall be used by determine the shear force. However, the force transmitted through a bearing pad shall be limited to the force that causes the pad to slip.

Hold-Down Devices shall be designed per AASHTO 3.10.9.6. This requires a minimum earthquake force to cause uplift on the substructure equal to 10 percent of the dead load reaction of the superstructure. Where such forces can be developed, the crossbeam, column and footing shall be designed to carry these earthquake forces.

For concrete superstructures built integrally with the substructure, the substructure elements shall be designed to carry their dead load plus all the elements below them including soil overburden as though they were suspended from the superstructure. (Seal not included). For this condition, the ultimate downward force shall be 1.0 (EQ + Uplift). For structures carried on elastomeric pads or where there is no positive vertical connection, the uplift force from the superstructure shall be neglected.

Post-tensioning Effects from Superstructure - EL

When cast-in-place, post-tensioned superstructures are constructed monolithic with the piers, the substructure design should take into account frame moments and shears caused by elastic shortening and creep of the superstructure upon application of the axial post-tensioning force at the bridge ends. Frame moments and shears thus obtained should be added algebraically to the values obtained from the primary and secondary moment diagrams applied to the superstructure.

When cast-in-place, post-tensioned superstructures are supported on sliding bearings at some of the piers, the design of those piers should include the longitudinal force from friction on the bearings generated as the superstructure shortens during jacking. When post-tensioning is complete, the full permanent reaction from this effect should be included in the governing AASHTO load combinations for the pier under design.

Wind Loads – WL and WS

Wind forces shall be applied to the substructure units in accordance with the loadings specified in AASHTO. Transverse stiffness of the superstructure may be considered, as necessary, to properly distribute loads to the substructure provided that the superstructure is capable of sustaining such loads. Vertical wind pressure, per AASHTO 3.8.2, shall be included in the design where appropriate, for example, on single column piers. Wind loads shall be applied through shear keys or other positive means from the superstructure to the substructure. Wind loads shall be distributed to the piers and abutments in accordance with the laws of statics. Transverse wind loads can be applied directly to the piers assuming the superstructure to act as a rigid beam. For large structures a more appropriate result might be obtained by considering the superstructure to act as a flexible beam on elastic supports.

7.1.4 Concrete Class for Substructure

The concrete class for the design of substructure units shall normally be Class 4000. This includes footings, pedestals, massive piers, columns, crossbeams, traffic barriers. Foundation seals will use Class 4000W.

Where retaining walls are connected directly to the bridge superstructure and color matching is important, consideration could be given to using Class 4000 in the retaining wall or using pigmented sealer in order that the concrete color will not vary from adjacent portions of the structure.

7.1.5 Foundation Seals

A concrete seal within the confines of a cofferdam permits construction of a pier footing and column in the dry. This type of underwater construction is practical to a water depth of approximately 50 feet.

Seal concrete is placed underwater with the use of a tremie. A tremie is a long pipe that extends to the bottom of the excavation and permits a head to be maintained on the concrete during placement. After the concrete has been placed and has obtained sufficient strength, the water within the cofferdam is removed. The weight of the seal concrete resists the hydrostatic pressure exerting force at the base of the seal. In Figure 7-5, some of the factors that must be considered in designing a seal are illustrated.

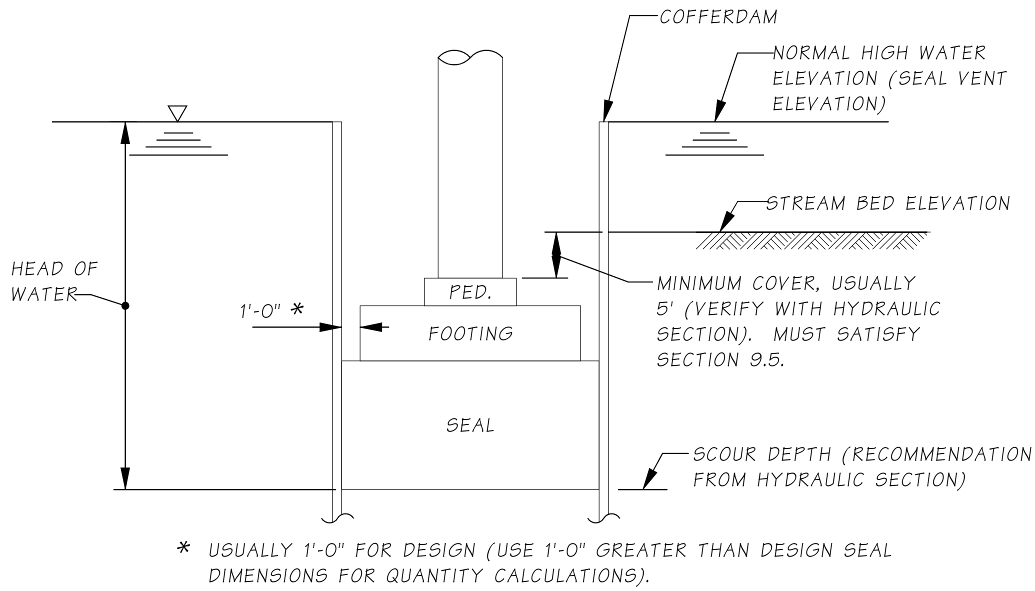


Figure 7-5

A. General Seal Criteria

The Normal High Water Elevation is defined as the highest water surface elevation that may normally be expected to occur during a given time period. This elevation, on the Hydraulics Data Sheet, is obtained from discussions with local residents or by observance of high water marks at the site. The normal high water is not related to any flood condition.

Seal Vent Elevation

The headquarters Hydraulics Section recommends a seal vent elevation in accordance with the following criteria:

1. Construction time period not known

   If the time period of the footing construction is not known, the vent elevation reflects the normal high water elevation that might occur at any time during the year.

2. Construction time period known.

   If the time period of the footing construction can be anticipated, the vent elevation reflects the normal high water elevation that might occur during this time period. (If the anticipated time period of construction is later changed, the Hydraulics Section shall be notified and appropriate changes made in the design.)

Scour Depth

The Headquarters Hydraulics Section determines the depth of the anticipated scour. The bottom of footing, or bottom of seal if used, shall be no higher than the scour depth elevation. After preliminary footing and seal thicknesses have been determined, the Bridge Designer shall review the anticipated scour elevation with the Hydraulics Section to ensure that excessive depths are not used.

Foundation Elevation Recommended in Geotechnical Report

Based on the results obtained from test borings at the site, the Geotechnical Engineer determines a foundation elevation, bearing capacity and settlement criteria. If other factors control, such as scour or footing cover, the final footing elevation should be adjusted as required.

Unusual Conditions

Unusual site conditions such as rock formations or deep foundations require special considerations in order to obtain the most optimum design. The proposed foundation design/construction should be discussed with both the Geotechnical Engineer and the Bridge Hydraulics Section prior to final plan preparation.

B. Spread Footing Seals

The Geotechnical Division will generally recommend whether a foundation seal may or may not be required for construction. Bearing loads are the column moments applied at the base of the footing and vertical load applied at the bottom of the seal. The seal is sized for the soil bearing, capacity, and Overturning Stability need only be checked at the base of the pier footing.

When A Seal is Required During Construction

If the footing can be raised without violating cover requirements, the bottom of the seal elevation shall be the lower of the scour elevation or the foundation elevation as recommended by the Geotechnical Engineer. Bottom of seal may be lower than the scour elevation or foundation elevation due to cover requirements. Spread footing final design will include the dead load weight of the seal.

When Seal May Not Be Required for Construction

Both methods of construction are detailed in the Plans when it is not clear if a seal is required for construction. The Plans must detail a footing with a seal and an alternate without a seal. The Plan quantities are based on the footing designed with a seal. If the alternate footing elevation is different from the footing with seal, it is also necessary to note on the plans the required changes in rebar such as length of column bars, increased number of ties, etc.

C. Pile Footing Seals

The top of footing, or pedestal, is set by the footing cover requirements. The bottom of seal elevation is based on the stream scour elevation determined by Hydraulics. A preliminary analysis is made using the estimated footing and seal weight, and the column moments and vertical load at the base of the footing to determine the number of piles and spacing. Seal size will be 1 foot 0 inches larger than the footing all around. If the seal is omitted during construction, the bottom of footing shall be set at the scour elevation and an alternate design is made. The following LFD Timber Pile Footing example re-printed in the BDM as a guideline and not intended to be used directly for design.

7.1.5.C.1 Timber Pile Footing Example

In no case shall the uplift exceed the weight of the material (buoyancy considered) surrounding the embedded portion of the pile, using a factor of safety of 1.25.

Example:

Head in feet on bottom of seal = 45.0 feet

40 Ton timber piles

Pile spacing 3-foot 3-inh centers each way

Unit submerged weight of soil $\leq$ 50 pounds per cubic foot

Unit weight water = 62.5 pounds per cubic foot fresh water; 64 pounds per cubic foot sea water

Factor of safety = 1.25

From chart, "Thickness of Foundation Seals," Appendix 9.7-A1, thickness of seal = 5.0 feet

Uplift per pile = (head) (Pile spacing)^2 (Unit weight water) minus (seal thickness) (Pile spacing)^2 (Unit weight seal concrete) plus (Volume embedded pile) (Unit weight seal concrete)

Uplift per pile = $45\ (3.25)^2\ (.0625) - (5.0)\ (3.25)^2\ (0.145) + 5.0\ (\frac{\pi}{4})\ (1.0)^2\ (0.145)$

Uplift per pile = $29.7 - 7.6 + 0.6 = 22.7^k$/pile

$$\text{Pile Extension} = \frac{\text{(Uplift per pile) (F.S.)}}{\text{(Pile spacing)}^2\ \text{(Unit submerges weight of soil)}}$$

$$\text{Pile Extension} = \frac{22.7^k\ (1.25)}{(3.25)^2\ (0.050)} = 53.7 \text{ feet (Say 54 feet)}$$

Must be shown on plans.

If it is not practical to drive piling this far, additional seal thickness must be added. Consult with your supervisor.

7.2 Foundation Modeling

7.2.1 General Modeling Concepts

Proper bridge modeling for seismic events is important to ensure that foundations remain stable and the substructure above ground remains ductile. Misinterpreted AASHTO Specifications and modeling errors can lead to a wide range of substructure sizes. Unrealistic models will likely produce unnecessary costs in substructure members. Extreme event forces will generally govern in regions of high seismic acceleration. Strength forces will generally govern in regions of low acceleration.

The bridge analysis will use unfactored loads to determine seismic load distribution. Unfactored loads and seismic response are then factored for design of substructure elements.

Liquefiable Soils

Soil liquefaction requires a separate bridge analysis used concurrently with a static soil analysis to check the design of substructure elements. Liquefied soil generally reduces the soil lateral resistance and increases the axial loads. Load cases with liquefied soils will generally govern for designs that consider deflection, such as bearings or expansion gaps. Large structures with independent segments and liquefiable soil must be investigated for seismic performance. Load cases with static soils will generally govern for design that considers forces (or strength limit states). Bridge design must check the foundation capacity and superstructure behavior for both soil conditions. Consult the Supervising Bridge Engineer if liquefiable soils lead to cost prohibitive designs.

Abutments

Most bridge models should include an effective abutment stiffness that accounts for expansion gaps and incorporates a realistic value for the embankment fill response. The abutment fill stiffness is nonlinear and is dependent upon on the soil properties. A longitudinal spring is not required for L-type abutments if seismic superstructure deflections do not impact the backwall.

Other structures such as stub abutments or L-type abutments with superstructures impacting the backwall should include a longitudinal spring for the abutment fill. Based on passive earth pressure tests and the force deflection results from large-scale abutment testing at UC Davis, the initial embankment fill stiffness is 20 Kip/in per foot of wall. The initial stiffness is adjusted proportional to the backwall/diaphragm height, as shown in Equation 7.2.1.1. Where, W_{bw} is the Width (ft) and H_{bw} is the Height (ft) of the backwall for L-type abutments, or the diaphragm for Stub abutments.

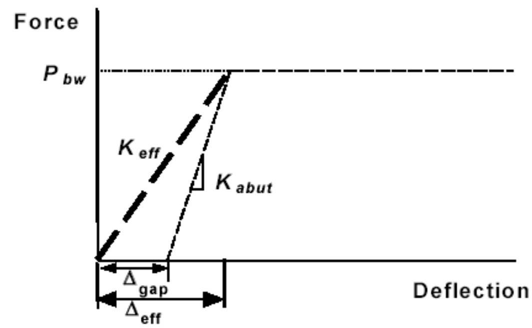
$$K_{abut} = \frac{20kip/in}{ft} \times W_{bw} \times \frac{H_{bw}}{5.5}$$

Equation 7.2.1-1

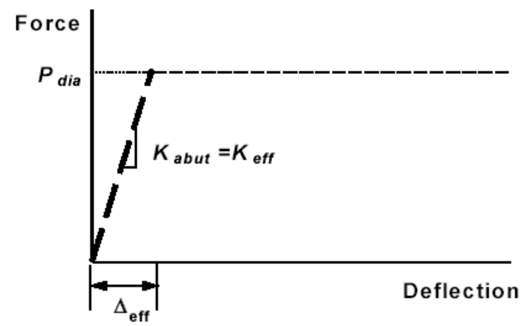
The passive pressure (P) resisting the movement at the abutment increases linearly with the displacement to a maximum passive pressure of 5.0 ksf, presented in Equation 7.2.1.2. The maximum pressure is based on the ultimate static force developed in the full scale abutment testing conducted at UC Davis [Maroney, 1995]. See CalTrans Seismic Design Criteria, Version 1.3, Feb. 2004 for more detailed information on abutment springs.

$$P_{force} = H_{bw} \times W_{bw} \times 5.0ksf \times \frac{H_{bw}}{5.5}$$

Equation 7.2.1-2



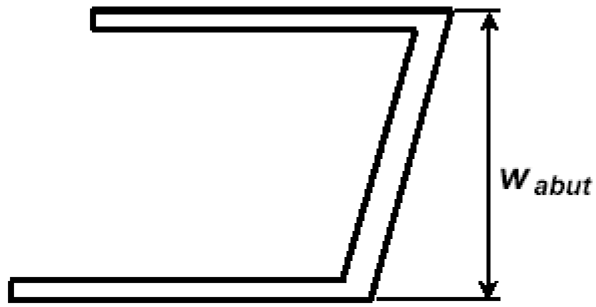
L-type Abutment Spring
Figure 7-6



Stub Abutment Spring
Figure 7-7

*where diaphragms are designed for passive pressure.

Abutment Height and Width
Figure 7-8



Abutment Width for Skewed Bridges
Figure 7-9

PY Curves and Soil Modulus

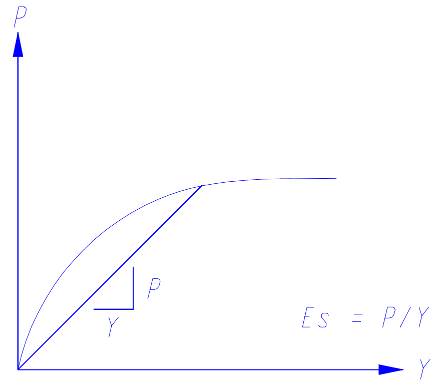
A finite element model may use non-linear springs based on PY curves to represent foundation response as shown in Figure 7.1.2-5. PY curves graph the relationship between the lateral soil resistance and the associated deflection of the soil. Generally, P stands for a force per unit length (of pile) such as kips per inch. Y is the corresponding horizontal deflection (of pile) in units such as inches.

Pile Model using a Set of Non-linear PY Curves
Figure 7-10

Node placement for springs should attempt to imitate the soil layers. Generally, the upper 1/3 of the pile in stiff soils has the most significant contribution to the lateral soil reaction. Springs in this region should be spaced at most 3 feet apart. Spacing of 2.5 feet has demonstrated results within 10% of Lpile output moment and shear. Springs for the lower 2/3 of the pile can transition to a much larger spacing. Stiff foundations in weak soils will transfer loads much deeper in the soil and more springs would be sensible.

Transverse and longitudinal springs must include group reduction factors to analyze the structure/soil response. Soil properties are modified in Lpile to account for Group Effects. Lpile then generates PY curves based on the modified soil properties and desired depths. See BDM Section 7.2.5 for Group Effects.

FEM programs will accept non-linear springs in a Force (F) vs. Deflection (L) format. P values in a PY curve must be multiplied by the pile length associated with the spring in the FEM. This converts a P value in Force/Length units to Force.



Secant Modulus Illustration
Figure 7-11

Soil Modulus - E_S

Soil Modulus is defined as the force per length (of a pile) associated with a soil deflection. As shown in Figure 7.1.2-6, E_S is a slope on the PY curve or P/Y. E_S is a secant modulus since the PY relationship is nonlinear and the modulus is a constant. The units are F/L per L or F/L^2, such as kips per square inch.

Subgrade Modulus - k_S

A closely related term is the Subgrade Modulus (or Modulus of Subgrade Reaction) provided in a Geotechnical Report. This is defined as the soil pressure associated with a soil deflection. The units are F/L^2 per L or F/L^3, such as kips per cubic inch.

7.2.2 Substructure Analysis Flow Chart

The following is a general description of the iterative process that converges to acceptable seismic design forces. See Figure 7-12 for a Substructure Seismic Flow Chart.

1. Build a Finite Element Model (FEM) in order to determine initial forces to substructure elements (EQ+DL). Assume the foundation springs are located at the ground line. A good initial support assumption for piles would be to add 10 feet to the column length in stiff soils and 15 feet to the column in soft soils. The same can be assumed for shafts. An alternate method is to use 85% of the fixed support reactions for the initial forces. If fixed supports are used for the initial foundation model, more iteration will be required to converge. Use the fully fixed forces for foundations in rock.

   Use multi-mode response spectrum analysis to generate initial Seismic Shear, Moment, & Axial Loads. Analysis will use unfactored loads. Factored analysis results are used for design.

2. Using initial model forces, determine a preliminary footing size, shaft size/length, or pile group arrangement for bridge supports.

3. The FEM and the soil/structure interaction analysis must agree or converge on soil/structure lateral response. In other words, moment and shear, deflection and rotation, and the spring values of the two programs should be within 5% or 10%. More iteration will provide convergence much less than 1%. The iteration process to converge is as follows:

   a. Apply the initial FEM loads (moment and shear) to a soil/structure lateral response program such as Lpile or S-Shaft.

   b. Calculate foundation spring values for the FEM.

c. Re-run the seismic analysis. The structural response will change. Check to insure the FEM results (M, V, Δ, θ, and spring values) in the transverse and longitudinal direction are within 5% of the previous run. This check verifies the linear spring soil response (calculated by the FEM) is close to the predicted nonlinear soil behavior (calculated by the soil response program). If the results of the FEM and the soil response program are different, recalculate springs and repeat steps (a) thru (c) until the two programs converge within 5%.

   If the fixed head boundary conditions are incorrectly assumed for a free head situation, the FEM's moment and shear may still converge to the soil response program's moment and shear. However, the soil response will be much stiffer (and incorrect). The deflections and rotations of the Soil response program will not match the FEM deflections and rotations since the boundary conditions do not match. See BDM Section 7.2.5.1.

   Designers should note the magnitude of shear and moment at the top of the shaft. The seismic design philosophy requires a plastic hinge in the substructure elements above ground (in the columns). If the column "zero" moment is close to a shaft head foundation spring, the FEM and Soil response program will not converge and plastic hinging might be below grade.

4. Revise the preliminary substructure or adjust the springs to be within structural and geotechnical limits.

Substructure Seismic Flow Chart
Figure 7-12

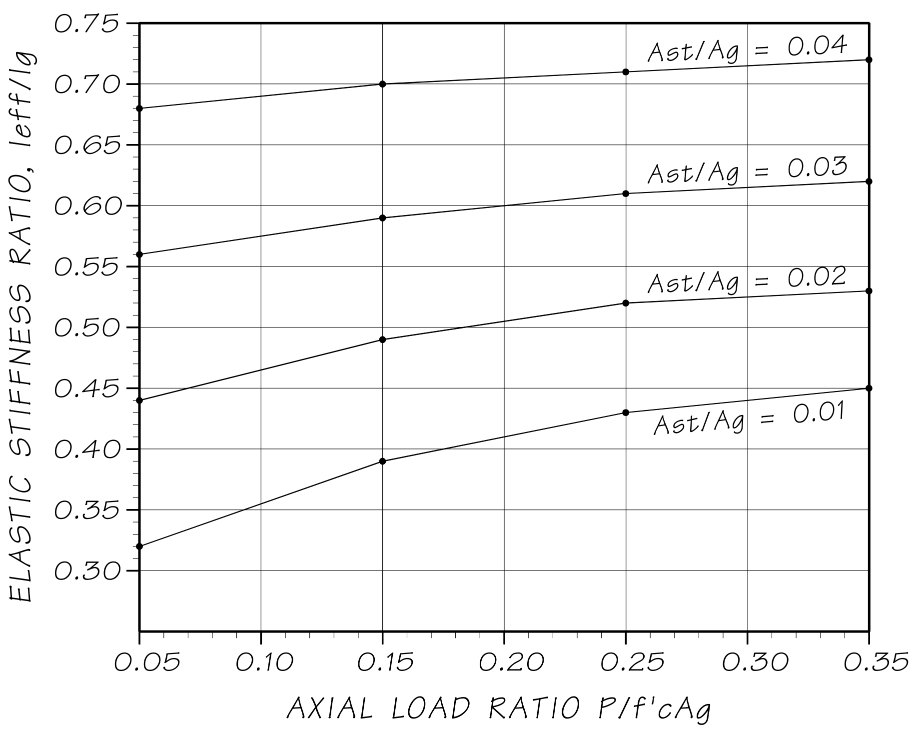
7.2.3 *Bridge Model Section Properties*

In general, gross section properties may be assumed for all FEM members, except concrete columns. The concrete unit weight for seismic analysis will be the same as for strength design, 160 pcf.

Cracked Properties for Columns

The overall seismic design goal is to achieve ductile performance and avoid bridge collapse. Columns analyzed with non-cracked properties form plastic hinges at higher stress levels, which minimize damage in an intermediate earthquake. (Minimal damage is spalling of concrete cover and small permanent deformations.) However, this assumption results in larger foundations than using the column cracked section. The seismic responses of a bridge analyzed with cracked sections have longer periods and reduce lateral forces and foundation size. The cracked column assumption will result in a higher potential damage for intermediate earthquakes, but is considered a reasonable risk. Both cracked and non-cracked column assumptions result in ductile performance using AASHTO response modification factors, R.

Concrete columns assume cracked section properties since plastic hinges are designed to form in these members. In general, column properties in FEM are rotated 90 degrees from the Global axis; and Beta angles are used to orient the column local axis to the Pier skew. In lieu of hand calculations, Figure 7-13 presents a stiffness ratio for round columns which may be used to approximate the effective moment of inertia, I_{eff} for all bridge columns. Figure 7-13 is a reprint from Figure 85, page 200, Seismic Retrofitting Manual for Highway Bridges, Pub. No. FHWA-RD-94-052, May 1995.



Effective Stiffness for Circular Bridge Columns
Figure 7-13

Torsional Properties

The torsional moment of inertia, for non-circular shapes is not the Polar Moment of Inertia (calculated by GDS). The sum of $bt^3/3$ may be used to calculate torsional stiffness of deck/I girder superstructures, where the girders are equivalent rectangular areas with the same height.

Shaft Properties

The moment of inertia for shafts will be based on the gross section (Ig) or non-cracked. The shaft concrete strength and construction methods lead to significant variation in shaft stiffness described as follows.

For a stiff substructure response:

- Use 1.5 f'c to calculate the modulus of elasticity. Since aged concrete will generally reach a strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a reasonable estimate for an increase in stiffness.
- Increase shaft Ig by adding 6" to the shaft diameter. The WSDOT shaft special provisions allow the contractor to increase the shaft diameter up to 6" to accommodate metric sizes.
- When permanent casing is specified, increase shaft Ig using the transformed area of a ¾" thick casing. Since the contractor will determine the thickness of the casing, ¾" is a conservative estimate for design.

For a soft substructure response,

- Use 0.85 f'c to calculate the modulus of elasticity. Since the quality of shaft concrete can be suspect when placed in water, the factor of 0.85 is an estimate for a decrease in stiffness.
- Use shaft Ig.
- When permanent casing is specified, increase shaft Ig using the transformed area of a 3/8" thick casing. Since the contractor will determine the thickness of the casing, 3/8" is a minimum estimated thickness for design.

7.2.4 Bridge Model Verification

As with any FEM, the designer should review the foundation behavior to ensure the foundation springs correctly imitate the known boundary conditions and soil properties. Watch out for mismatch of units!

All finite element models must have dead load static reactions verified and boundary conditions checked for errors. The static dead loads (DL) must be compared with hand calculations or another program's results. For example, span member end moment at the supports can be released at the piers to determine simple span reactions. Then hand calculated simple span DL or PGsuper DL and LL is used to verify the model.

Crossbeam behavior must be checked to ensure the superstructure DL is correctly distributing to substructure elements. A bridge line model concentrates the superstructure mass and stresses to a point in the crossbeam. Generally, interior columns will have a much higher loading than the exterior columns. To improve the model, crossbeam I_{gross} should be increased to provide the statically correct column DL reactions. This may require increasing I_{gross} by about 1000 times. Many times this is not visible graphically and should be verified by checking numerical output.

Seismic analysis may also be verified by hand calculations. Hand calculated fundamental mode shape reactions will be approximate; but will insure design forces are of the same magnitude, (see AASHTO 4.7.4.3.2). FEM results must include enough mode shapes such that the total percent mass participation must be in excess of 90% for the transverse and longitudinal directions.

Designers should note that additional mass might have to be added to the bridge FEM for seismic analysis. For example, traffic barrier mass and crossbeam mass beyond the last column at piers may contribute significant weight to a two-lane or ramp structure.

7.2.5 Deep Foundation Modeling

A designer must assume a foundation support condition that best represents the foundation behavior. Deep foundation springs attempt to imitate the non-linear lateral behavior of several soil layers interacting with the deep foundation. The bridge FEM then uses the springs to predict the seismic structural response.

There are two accepted methods to model deep foundation lateral response. The Bridge and Structures Office prefers the first method. This models deep foundations by using linear support springs or a {6x6} matrix, if a cross couple coefficient is required. The linear springs are generated to reproduce the non-linear behavior of the soil and foundation at the maximum loading. Soil response programs, such as Lpile or S-Shaft, analyze the non-linear soil response. The results are then used to determine the linear springs.

The second method attaches non-linear springs along the length of deep foundation members in a model. See PY Curves in BDM Section 7.2.1. This method has the advantage of solving the superstructure and substructure seismic response simultaneously. The soil springs must be nonlinear PY curves and represent the soil/structure interaction.

Models using linear soil springs that are not based on non-linear soil-structure interaction are generally considered inaccurate for soil response/element stress and are not acceptable.

Spring Location

The preferred location for a foundation spring is at the ground line, which includes the column mass in the seismic analysis. For design, the column forces are provided by the FEM and the foundation forces are provided by the Geotech program. Springs may be located at the top of the column. However, the seismic analysis will not include the mass of the columns. The advantage of this location is the soil/structure analysis includes both the column and foundation design forces.

Designers should be careful to match the geometry of the FEM and Geotech program. If the location of the foundation springs (or node) in the FEM does not match the location input to the Geotech program, the two programs may not converge.

A. Boundary Conditions

To calculate spring coefficients, the designer must first identify the predicted shape, or direction of loading, of the foundation member where the spring is located in the bridge model. This will determine if one or a combination of two boundary conditions apply for the transverse and longitudinal directions of a support.

A fixed head boundary condition occurs when the FOUNDATION element is in double curvature where translation without rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the opposite direction of applied moment. This is a common assumption applied to both directions of a rectangular pile group that supports a footing pile cap.

A free head boundary condition is when the FOUNDATION element is in single curvature where translation and rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the same direction as the applied moment. Most large diameter shaft designs will have a single curvature below ground line and require a free head assumption. The classic example of single curvature is a single column on a single shaft. In the transverse direction, this will act like a flagpole in the wind, or free head. What is not so obvious is the same shaft will also have single curvature in the longitudinal direction (below the ground line), even though the column exhibits some double curvature behavior. Likewise, in the transverse direction of multi-column piers, the columns will have double curvature (frame action). The shafts will generally have single curvature below grade and the free head boundary condition applies. The boundary condition for large shafts with springs placed at the ground line will be free head in most cases.

The key to determine the correct boundary condition is to resolve the correct sign of the moment and shear at the top of the shaft (or point of interest for the spring location). Since multi-mode results are always positive (CQC), this can be worked out by observing the seismic moment and shear diagrams for the structure. If the sign convention is still unclear, apply a unit load in a separate static FEM run to establish sign convention at the point of interest.

The correct boundary condition is critical to the seismic response analysis. For any type of soil and a given foundation loading, a fixed boundary condition will generally provide soil springs four to five times stiffer than a free head boundary condition.

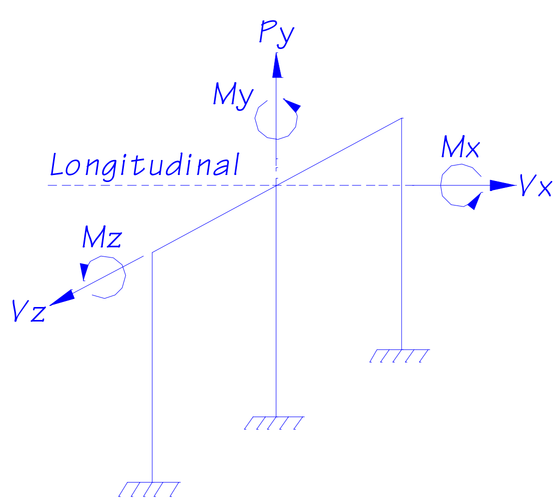
B. Linear Spring Calculation

The first step to calculate a foundation spring is to determine the shear and moment in the structural member where the spring is to be applied in the FEM. Foundation spring coefficients should be based on the maximum shear and moment from the applied longitudinal OR transverse seismic loading. The combined load case (1.0L and 0.3T) should be assumed for the design of structural members, and NOT applied to determine foundation response. For the simple case of a bridge with no skew, the longitudinal shear and moment are the result of the seismic longitudinal load, and the transverse components are ignored. This is somewhat unclear for highly skewed piers or curved structures with rotated springs, but the principle remains the same.

If Lpile is used for foundation analysis, group effects will require the geotechnical properties to be reduced before the spring values calculated. The group effects generally will be different transversely and longitudinally. The exception would be a single column with a single shaft.

1. Matrix Coordinate Systems

The Global coordinate systems used to demonstrate matrix theory are usually similar to the system defined for substructure loads in BDM Section 7.1.3, and is shown in Figure 7-14. This is also the default Global coordinate system of GTStrudl. This coordinate system applies to this BDM Section to establish the sign convention for matrix terms. Note vertical axial load is labeled as P, and horizontal shear load is labeled as V.



Standard Global Matrix
Figure 7-14

a) **Matrix Coefficient Definitions**

The stiffness matrix containing the spring values and using the standard coordinate system is shown in Figure 7-15. The sign of the K34 and K16 terms must be determined based on the sign convention. In this case, K34 will be negative. For detailed information on cross-couple terms, see Lateral Springs using Fixed Head Boundary Conditions.

The coefficients in the stiffness matrix are generally referred using several different terms. Coefficients, spring or spring value are equivalent terms. Lateral springs are springs that resist lateral forces. Vertical springs resist vertical forces.

$$\begin{Bmatrix} & Vx & Py & Vz & Mx & My & Mz \\ Vx & K11 & 0 & 0 & 0 & 0 & K16 \\ Py & 0 & K22 & 0 & 0 & 0 & 0 \\ Vz & 0 & 0 & K33 & K34 & 0 & 0 \\ Mx & 0 & 0 & K43 & K44 & 0 & 0 \\ My & 0 & 0 & 0 & 0 & K55 & 0 \\ Mz & K61 & 0 & 0 & 0 & 0 & K66 \end{Bmatrix} \times \begin{Bmatrix} Disp. \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z \end{Bmatrix} = \begin{Bmatrix} Force \\ Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz \end{Bmatrix}$$

Standard Global Matrix

Figure 7-15

Where the linear spring constants or K values are defined as follows using the Global Coordinates:

K11 = $+Vx_{app}/+\Delta x$ = Kip/in = Longitudinal Lateral Stiffness

K22 = AE/L = Kip/in = Vertical or Axial Stiffness

K33 = $-Vz_{app}/-\Delta z$ = Kip/in = Transverse Lateral Stiffness

K44 = $+Mx_{app}/+\theta x$ = K in/rad = Transverse Bending or Moment Stiffness

K55 = JG/L = K in/rad = Torsional Stiffness

K66 = $+Mz_{app}/+\theta z$ = K in/rad = Longitudinal Bending or Moment Stiffness

K34 = $-Vz_{ind}/+\theta_x$ = Kip /rad = Transverse Lateral Cross-couple term (Fixed Head only)

K16 = $+Vx_{ind}/+\theta_z$ = Kip /rad = Longitudinal Lateral Cross-couple term (Fixed Head only)

K43 = $+Mx_{ind}/-\Delta z$ = Kip /rad = Longitudinal Moment Cross-couple term (Fixed Head only)

K61 = $+Mz_{ind}/+\Delta x$ = Kip /rad = Transverse Moment Cross-couple term (Fixed Head only)

Diagonal Terms

The linear lateral spring constants along the diagonal represent a point on a non-linear soil/structure response curve. When calculating the spring values, the shear and moment must be applied at the same time to capture the non-linear response of the soil/structure interaction at the desired loading. These are linear approximations of the soil response for all loading applied to a structure. The springs are only accurate for the maximum loading and less accurate for loads less than maximum. This is considered acceptable for Strength and Extreme Event design.

The vertical and torsional constants (properties) are calculated based on statics.

b) Vertical Springs: K22

Vertical spring constants can be calculated from the following three assumptions. See Figure 7.2.5.2.3 and the following definitions. REF: Page 6-30, Seismic Design of Highway Bridges Workshop Manual, Pub. No. FHWA-IP-81-2, Jan 1981.

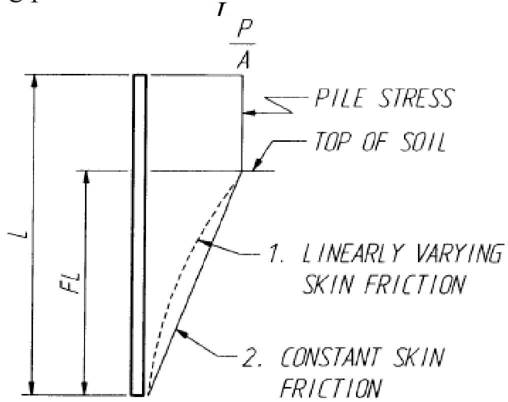
A = Cross sectional area

E = Young's modulus

L = Length of pile

F = Fraction of pile embedded

Point bearing piles: $K22 = \frac{AE}{L}$



Showing Pile Stress

Figure 7-15b

Friction piles:

1. Linearly varing skin friction: $K22 = 3\frac{AE}{L}$

2. Constant skin friction: $K22 = 2\frac{AE}{L}$

Partially embedded piles (in the soil):

1. Linearly varing skin friction: $K22 = \frac{AE}{\left(1-\frac{F}{2}\right)L}$

2. Constant skin friction: $K22 = \frac{AE}{\left(1-\frac{2F}{3}\right)L}$

c) **Torsional Springs: K55**

The S-Shaft program calculates acceptable torsional spring values for shafts and may be used for foundation springs. In general, torsional spring constants for individual piles are based on the strength of the pile. The statics equation for torsional resistance is given below.

$K55 = M/\phi = T/\phi = JG/L$

where, $G = 0.4\ E$

J = Torsional Moment of Inertia

L = Length of pile

d) **Lateral Springs using Fixed Head Boundary Condition**

If the shear and moment are OPPOSING where the spring is located, a fixed head boundary condition is required to model the loaded foundation in a finite element model.

Since applying load to a fixed end results in no reaction, a soil/structure interaction analysis will generally analyze the shear and moment simultaneously as a free head. Using the soil response results, a cross-couple correction term will be required in a FEM to produce the induced moment in the element modeling the fixed head condition. If accurate stresses in fixed head element are not required, the cross-couple term may be omitted.

There are two ways to model fixed head pile group. The most common method for a column footing is to use a group spring to model a group of piles or shafts as one set of springs. This method uses six linear springs to represent the foundation behavior. Lateral loads are resisted by Cross-couples terms do not apply and individual pile loads must be calculated from the FEM results.

The second method would be to model the individual piles. This is more helpful for analyzing local stresses in the foundation cap element and for each pile. Cross-couple terms may be included and individual pile loads are generated in the FEM.

Off-diagonal Terms (Cross-couples)

Cross-couple springs will not be symmetric for non-linear modeling foundation modeling. Since finite element programs will use matrix multiplication to generate reactions, doing the math is the easy way to show the effect of cross-couple terms. Note that K16 and K34 terms will have opposite signs.

The longitudinal reactions are:

$Vx = K11 \times \Delta_x + K16 \times \theta_z$ and $Mz = K61 \times \Delta_x + K66 \times \theta_z$

The transverse reactions are:

$Vz = K33 \times \Delta_z + K34 \times \theta_x$ and $Mx = K43 \times \Delta_z + K44 \times \theta_x$

For a true fixed head boundary condition (translation only) in the X and Z directions, there will be no rotation about the X and Z axis. θ_x and θ_z will be zero. This means the K34 and K16 cross-couple terms will not affect the shear reactions. Likewise, the K66 and K44 rotational terms zero out and do not effect the moment reaction. This leaves the K61 and K43 cross-couple terms to generate induced moments based on the deflections in the X and Z directions. Designers should note, the cross-couple moments are applied to a fixed footing element and are resisted axially by the piles. This affects the local stress in the footing and axial loading of the pile much more than the column moment and shear, which is usually the primary focus for design.

Modeling real life features may be somewhat different than the theoretically true fixed condition. The top of a column at the superstructure or some pile and shaft applications may have opposing shear and moment, however the moment may be much less than the theoretical induced free head moment value. In other words, there may be significant rotations that need to be accounted for in the spring modeling. Designers need to be aware of this situation and use engineering judgment. The FEM would have rotations about the X and Z axis. θ_x and θ_z will NOT be zero and both the cross-couples terms and rotational springs may significantly affect the analysis.

1) Longitudinal Springs

The following lateral spring sections describe the method to calculate a lateral, rotational, and cross couple springs for a theoretically fixed application. Axial load (Py), shear (Vx, Vz), and moment (Mx, Mz) would be provided by a FEM analysis that includes static dead load. A standard global coordinate system and sign convention applies.

Lateral Spring: K11 = $Vx_{app}/\Delta x$

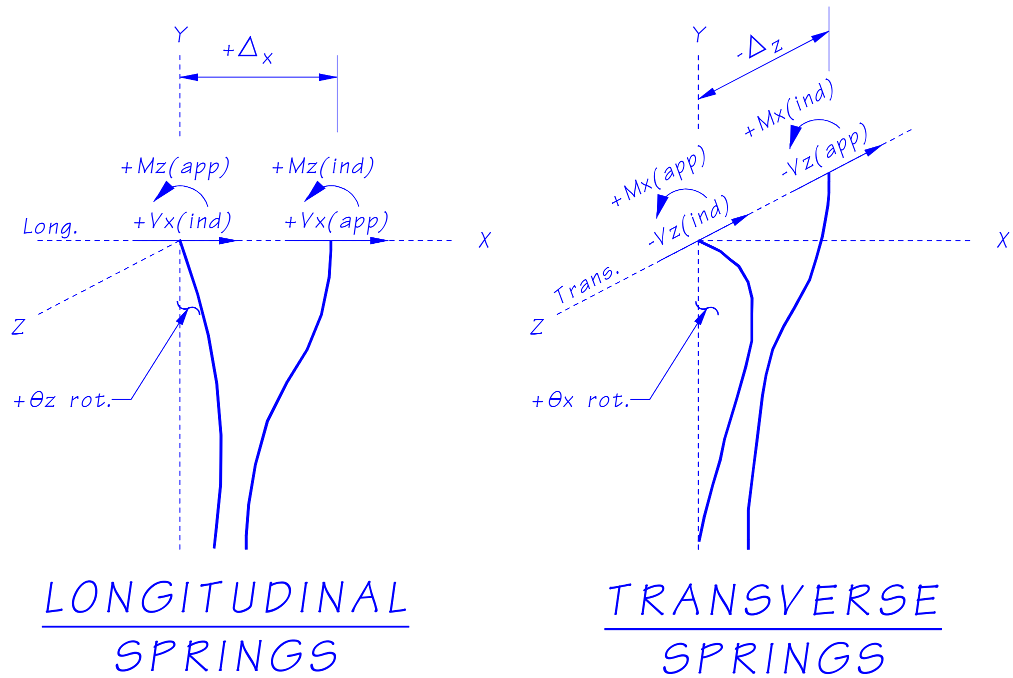
A longitudinal lateral spring is calculated by applying the longitudinal shear (Vx) and axial load (Py) in a soil response program with the rotation at the top equal to zero. The spring value is the applied shear (Vx_{app}) divided by the resulting deflection (Δx). The shear is positive and the deflection is positive. Therefore, K11 is positive.

Rotational Spring: K66 = $Mz_{app}/\theta z$

A longitudinal rotational spring is calculated by applying the longitudinal moment (Mz) and axial load (Py) in a soil response program with the translation at the top equal to zero. The spring value is the applied moment (Mz_{app}) divided by the resulting rotation (θz). The moment is positive and the rotation is positive. Therefore, K66 is positive.

Cross-couple Spring: K16 = Vx_{ind}/θ_z

K11 and K66 alone do not predict the shape or reaction of the foundation element. The cross-couple term K16 will add a shear force to correct the applied moment deflection. The spring value is the induced shear (Vx_{ind}) divided by the associated rotation (θz). The shear is positive and the rotation is positive. Therefore, K16 is positive.



Fixed Head Springs
Figure 7-16

2) Transverse Springs

The process is the same to calculate lateral K33, rotational K44, and cross-couple K34. The significance of repeating these steps is to show the significance of the sign convention. For the transverse springs, the sign of the shear and moment are the OPPOSITE. They work against each other. For this sample, the shear (-Vz) is negative and therefore Mx has to be positive.

Lateral Spring: K33 = $Vz_{app}/\Delta z$

A transverse lateral spring is calculated by applying the transverse shear (-Vz) and axial load (Py) in a soil response program with the rotation at the top equal to zero. The spring value is the applied shear ($-Vz_{app}$) divided by the resulting deflection (-Δz). The shear is negative and the deflection is negative. Therefore, K33 is positive.

Rotational Spring: K44 = $Mx_{app}/\theta x$

A transverse rotational spring is calculated by applying the transverse moment (Mx) and axial load (Py) in a soil response program with the translation at the top equal to zero. The spring value is the applied moment (Mx_{app}) divided by the resulting rotation (θx). The moment is positive and the rotation is positive. Therefore, K44 is positive.

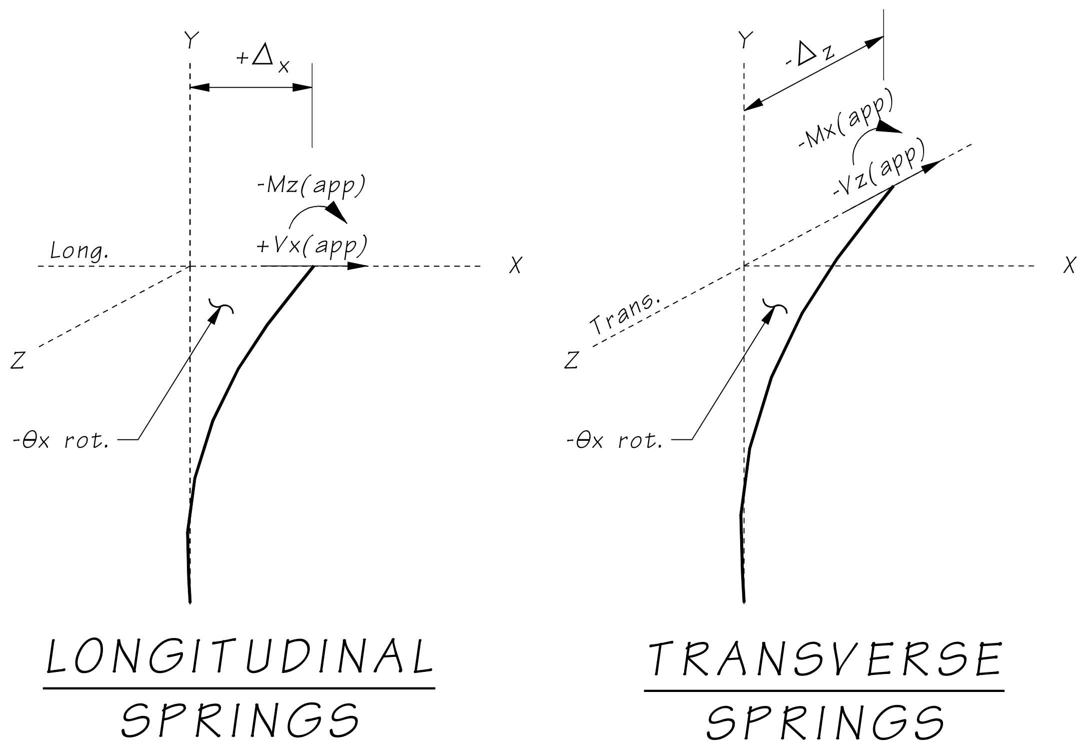
Cross-couple Spring: K34 = $-Vz_{ind} /\theta_x$

The cross-couple term K34 will add a shear force to correct the applied moment deflection. The spring value is the induced shear ($-Vz_{ind}$) divided by the associated rotation (θx). The shear is negative and the rotation is positive. Therefore, K34 is negative. If Mx were assumed negative, Vz would be positive and the K34 term would still be negative due to the sign convention of the coordinates. This indicates that the direction of loading does not matter to establish the sign of the cross-couple term.

e. Lateral Spring using Free Head Boundary Condition

If the shear and moment do not oppose each other a free head boundary condition is assumed to analyze a geotechnical response. Spring calculation for this boundary condition is somewhat quicker since only one geotechnical analysis is required and cross-couple terms do not apply. Spring values are calculated from the results of applying moment and shear at the same time and in the same direction to the foundation element. The lateral springs are simply V/Δ and the rotational springs are M/θ. Note when the free head shear and moment are working in the same direction, the spring values will always be positive.

Since cross-couple terms are not required, a matrix is not necessary to input spring coefficients into the finite element bridge model. Linear spring supports will yield the same results as a linear matrix.



Free Head Springs
Figure 7-17

f. Combination Free and Fixed Head Boundary Condition

An example of this case would be a single row of piles that support an abutment footing cap (at grade) with the superstructure supported on elastomeric bearings. In the transverse direction, the boundary condition is fixed head and would require a cross-couple term. In the longitudinal direction, the boundary condition is free head and the cross-couple term is zero as shown in Figure 7-18.

$$
\begin{array}{cc}
 & \begin{array}{cccccc} Vx & Py & Vz & Mx & My & Mz \end{array} \\
\begin{array}{c} Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz \end{array} &
\left\{\begin{array}{cccccc}
K11 & 0 & 0 & 0 & 0 & 0 \\
0 & K22 & 0 & 0 & 0 & 0 \\
0 & 0 & K33 & K34 & 0 & 0 \\
0 & 0 & K43 & K44 & 0 & 0 \\
0 & 0 & 0 & 0 & K55 & 0 \\
0 & 0 & 0 & 0 & 0 & K66
\end{array}\right\}
\end{array}
\times
\begin{array}{c} Disp. \\ \left\{\begin{array}{c} \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z \end{array}\right\} \end{array}
=
\begin{array}{c} Force \\ \left\{\begin{array}{c} Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz \end{array}\right\} \end{array}
$$

Figure 7-18

The moment applied to the abutment about the longitudinal axis (Mx) is resisted axially by the piles. Therefore, the moment applied to the pile (Mx_{app}) is zero. Since the induced shear (Vz_{ind}) is also zero, the K34 term ($Vz_{ind}/\theta x$) cross-couple term should be calculated using $Mx_{ind}/\Delta z$; which is K43.

C. Group Effects

If the S-Shaft program is used for soil/structure interaction analysis, the lateral group effect is calculated based on Wedge Theory, and included in the results.

When a foundation analysis uses Lpile or an analysis using PY relationships, the Geotechnical Report will provide transverse and longitudinal multipliers that are applied to the PY curves. This will reduce the pile resistance in a linear fashion. The reduction factors for lateral resistance due to the interaction of deep foundation members is provided in the WSDOT Geotechnical Design Manual, Section 8.12.2.5.

D. Pile Footing M**atrix** using Lpile and GTStrudl:

A matrix with cross-couple terms is a valid method to model pile supported footings. The analysis assumes the piles will behave similar to a column fixed at the bottom (in the soil) with lateral translation only at the top (no rotation). This requires Fixed Head Boundary Condition to calculate spring values.

The Lpile program will solve for non-linear soil results for individual piles. See Group Effects in BDM section 7.2.5.C to reduce the soil properties of a pile in a group in both the transverse and longitudinal directions. This sample matrix calculates a foundation spring for an individual pile.

If a pile group has a large number of piles, the GPILE computer program is available to generate a spring matrix for the group. The program also computes individual pile loads and deflections from input loads. The output will contain a SEISAB {6 x 6} stiffness matrix. GTStrudl or SAP matrices have the same coefficients with a different axis orientation for the pile group.

The pile spring requires eight pile stiffness terms for a matrix as discussed in BDM Section 7.2.5.B. The following sample calculations discuss the lateral, longitudinal, and cross-couple spring coefficients for a GTStrudl local coordinate system. Axial and torsion springs, two of the eight coefficients, will not be discussed in this section.

The maximum FEM transverse and longitudinal seismic loads (Vy, Mz, Vz, My and axial Px) provide two loads cases for analysis in Lpile. The Lpile results of these two load cases will be used to calculate lateral, longitudinal, and cross-couple spring coefficients.

This sample calculation assumes there are no group effects, no differential loading between the transverse and longitudinal directions. Therefore, Vy = Vz and My = Mz. A standard global coordinate system is assumed for the bridge. This sample will also assume a GTStrudl element is used to provide the foundation spring, which requires a different local axis coordinate system to input matrix terms, as shown in Figure 7-19. When the coordinate system changes, the sign convention of shear and moment also will change. This will be expressed in a 6x6 matrix by changing the location of the spring values and in sign of any cross-couple terms.

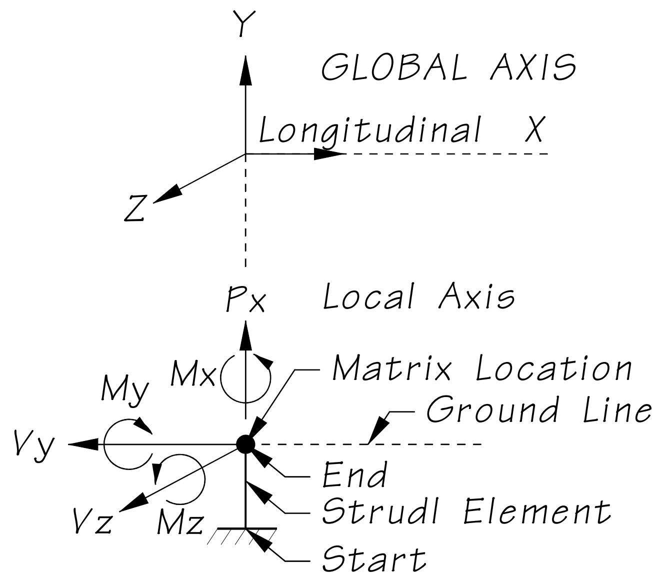


Figure 7-19

The locations of GTStrudl matrix terms are shown in Figure 7-20. The displacements are local and this requires the spring coefficients to be moved to produce the correct local reactions. The X axis is the new vertical direction. The Y axis is the new longitudinal direction. The spring coefficient definitions and notation remains the same as defined in BDM Section 7.2.5. Note the shift in diagonal terms and locations of the cross-couple terms. K16 and K61 are now **negative**.

$$\begin{array}{c} \\ Px \\ Vy \\ Vz \\ Mx \\ My \\ Mz \end{array} \begin{array}{c} \begin{array}{cccccc} Px & Vy & Vz & Mx & My & Mz \end{array} \\ \left\{\begin{array}{cccccc} K22 & 0 & 0 & 0 & 0 & 0 \\ 0 & K11 & 0 & 0 & 0 & K16 \\ 0 & 0 & K33 & 0 & K34 & 0 \\ 0 & 0 & 0 & K55 & 0 & 0 \\ 0 & 0 & K43 & 0 & K44 & 0 \\ 0 & K61 & 0 & 0 & 0 & K66 \end{array}\right\} \end{array} \times \left\{\begin{array}{c} Disp. \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z \end{array}\right\} \equiv \left\{\begin{array}{c} Force \\ Px \\ Vy \\ Vz \\ Mx \\ My \\ Mz \end{array}\right\}$$

Figure 7-20

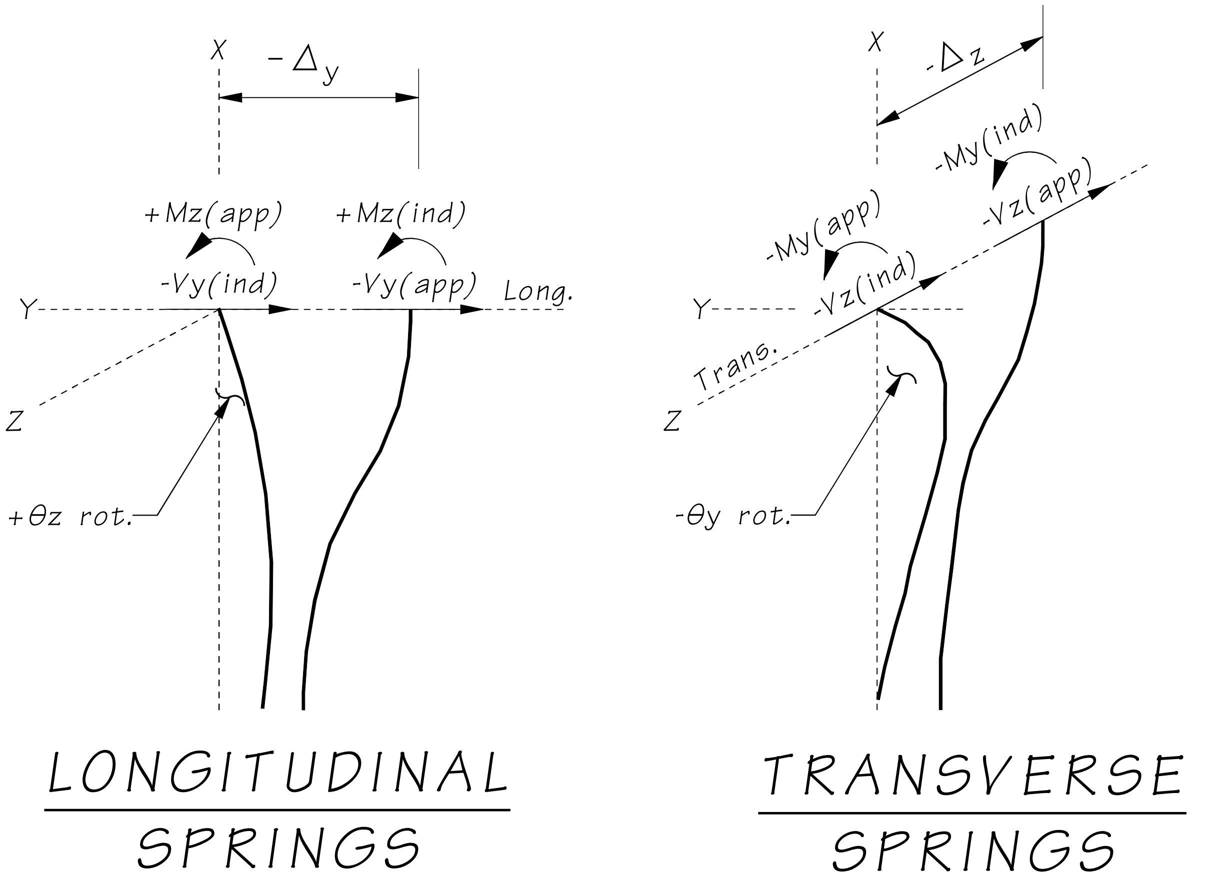


Figure 7-21

Where the linear spring constants or K values are defined as:

K11	$= -Vy/-\Delta y$	= positive	= Kip/in	= Longitudinal Lateral Stiffness
K22	$= AE/L$	= positive	= Kip/in	= Vertical or Axial Stiffness
K33	$= -Vz/-\Delta z$	= positive	= Kip/in	= Transverse Lateral Stiffness
K44	$= -My/-\theta y$	= positive	= K in/rad	= Transverse Bending or Moment Stiffness
K55	$= JG/L$	= positive	= K in/rad	= Torsional Stiffness
K66	$= Mz/\theta z$	= positive	= K in/rad	= Longitudinal Bending or Moment Stiffness
K34	$= -Vz_{ind}/-\theta\ y$	= positive	= Kip /rad	= Trans. shear X-couple term (Fixed Head only)
K16	$= -Vy_{ind}/\theta z$	= negative	= Kip /rad	= Long. shear X-couple term (Fixed Head only)
K43	$= -My_{ind}/-\Delta z$	= positive	= Kip /rad	= Long. Moment X-couple term (Fixed Head only)
K61	$= +Mz_{ind}/-\Delta y$	= negative	= Kip /rad	= Trans. Moment X-couple term (Fixed Head only)

Lateral Lpile Springs: K11 and K33

The loading should apply the lateral load (shear = V) and restrain the top against rotation (slope = 0).

Boundary condition code	=	2
Lateral load at the pile head	=	0.250D+05 lbs
Slope at the pile head	=	0.000D+00 in/in
Axial load at the pile head	=	0.758D+05 lbs = P

X	Deflection	Moment	Shear	Soil Reaction	Total Stress	Flexural Rigidity
In	In	Lbs-In	Lbs	Lbs-In	Lbs/In**2	Lbs-In**2
************	************	************	************	************	************	************
0.00	0.267D+01	-0.383D+07	0.250D+05	0.000D+00	0.270D+05	0.392D+11

$-Vy_{app}$ or $-Vz_{app}$ = 25 kip

$-\Delta y$ or $-\Delta z$ = 2.67”

K11 = -Vy/-Δy and is equal to K33 = -Vz/-Δz = $\frac{-25kip}{-2.67in} = 9.36\frac{kip}{in}$

Note the induced moment values for cross-couple calculation are $+Mz_{ind}$ or $-My_{ind}$ = 3830 k-in.

Moment Lpile Springs: K44 and K66

The loading should apply a moment (M) and restrain the pile top against translation (deflection = 0).

Boundary condition code	=	4
Deflection at the pile head	=	0.000D+00 in
Moment at the pile head	=	0.391D+07 in-lbs
Axial load at the pile head	=	0.103D+06 lbs

X	Deflection	Moment	Shear	Soil Reaction	Total Stress	Flexural Rigidity
In	In	Lbs-In	Lbs	Lbs-In	Lbs/In**2	Lbs-In**2
************	************	************	************	************	************	************
0.00	0.000D+00	0.391D+07	0.189D+05	0.000D+00	0.281D+05	0.392D+11

$+Mz_{app}$ or $-My_{app}$ = 391 K-in

$+\theta z$ or $-\theta y$ = 0.00845 rad

K44 = +Mz/+θz and is equal to K66 = -My/-θy = $\frac{-391kip}{-0.00845rad} = 46{,}272\frac{kip}{rad}$

Note the induced shear values for cross-couple calculation are $-Vy_{ind}$ or $-Vz_{ind}$ = 18.9 kip

Cross Couple Springs: K34 and K16

Given $-Vy_{ind}$ or $-Vz_{ind}$ = 18.9 kip and $+\theta z$ or $-\theta y$ = 0.00845 rad

Given $+Mz_{ind}$ or $-My_{ind}$ = 3830 k-in and $-\Delta y$ or $-\Delta z$ = 2.67”

Transverse K34 = -Vz_{ind} /-θy $= \frac{-18.9 kip}{-0.00845 rad} = 2236 \frac{kip}{rad}$

Transverse K43 = -My_{ind} /-Δz $= \frac{-3830\ kip\ in}{-2.67\ in} = 1434 \frac{kip}{rad}$

Use average value for cross-couple: K34 = K43 $= \frac{2236 + 1434}{2} = 1835 \frac{kip}{rad}$

Longitudinal K16 = -Vy_{ind} /+θz $= \frac{-18.9 kip}{0.00845 rad} = -2236 \frac{kip}{rad}$

Longitudinal K61 = +Mz_{ind} /-Δy $= \frac{3830\ kip\ in}{-2.67\ in} = -1434 \frac{kip}{rad}$

Use average value for cross-couple: K16 = K61 $= \frac{-2236 + -1434}{2} = -1835 \frac{kip}{rad}$

$$\left\{\begin{array}{ccccccc} & Px & Vy & Vz & Mx & My & Mz \\ Px & K22 & 0 & 0 & 0 & 0 & 0 \\ Vy & 0 & 9.36 & 0 & 0 & 0 & -1835 \\ Vz & 0 & 0 & 9.36 & 0 & 1835 & 0 \\ Mx & 0 & 0 & 0 & K55 & 0 & 0 \\ My & 0 & 0 & 1835 & 0 & 46272 & 0 \\ Mz & 0 & -1835 & 0 & 0 & 0 & 46272 \end{array}\right\} \times \left\{\begin{array}{c} Disp. \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z \end{array}\right\} = \left\{\begin{array}{c} Force \\ Px \\ Vy \\ Vz \\ Mx \\ My \\ Mz \end{array}\right\}$$

7.2.6 Spread Footing Modeling

For a first trial footing configuration, Strength column moments or column plastic hinging moments may be applied to generate footing dimensions. Soil spring constants are developed using the footing plan area, embedment depth, Poisson's ratio ν, shear modulus G. Spring constants for shallow rectangular footings are obtained by modifying circular footing theory using the following Equation. This method for calculating footing springs is referenced in FHWA-IP-87-6, Section 7.2.4A, page 140.

$K = \alpha\beta K_o$

K = Rotational or Lateral spring

K_o = Stiffness coefficient for the equivalent circular footing, see Figure 7-22. These values are calculated using an equivalent circular footing radius. See Figure 7-32

α = Foundation shape correction factor, see Figure 7-24

β = Embedment factor, see Figure 7-25

Displacement Degree-of-Freedom	**K_O**
vertical translation	4GR/1*v*
horizontal translation	8GR/2-*v*
torsional rotation	$16GR^3/3$
rocking rotation	$8GR^3/3(1-v)$

Stiffness Coefficients

Figure 7-22

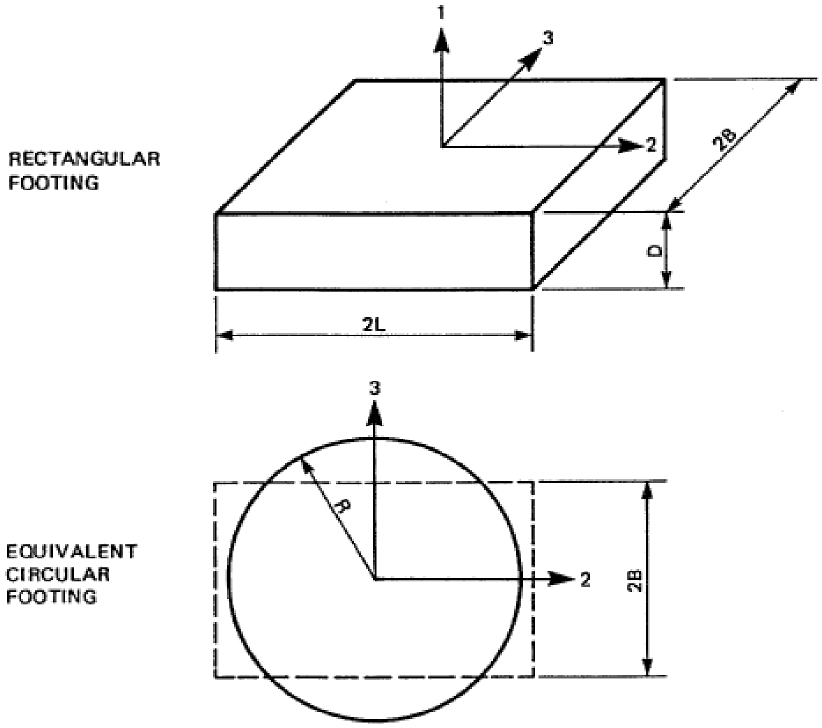


Figure 7-23

Figure 7-23 describes the orientation for used to calculate the equivalent radius values. The global Bridge orientation would be: Axis 2 = Axis Z or transverse to the bridge. The equations for the equivalent circular footing radius (R) are given below. The appropriate equivalent radius is used to calculate the desired spring value.

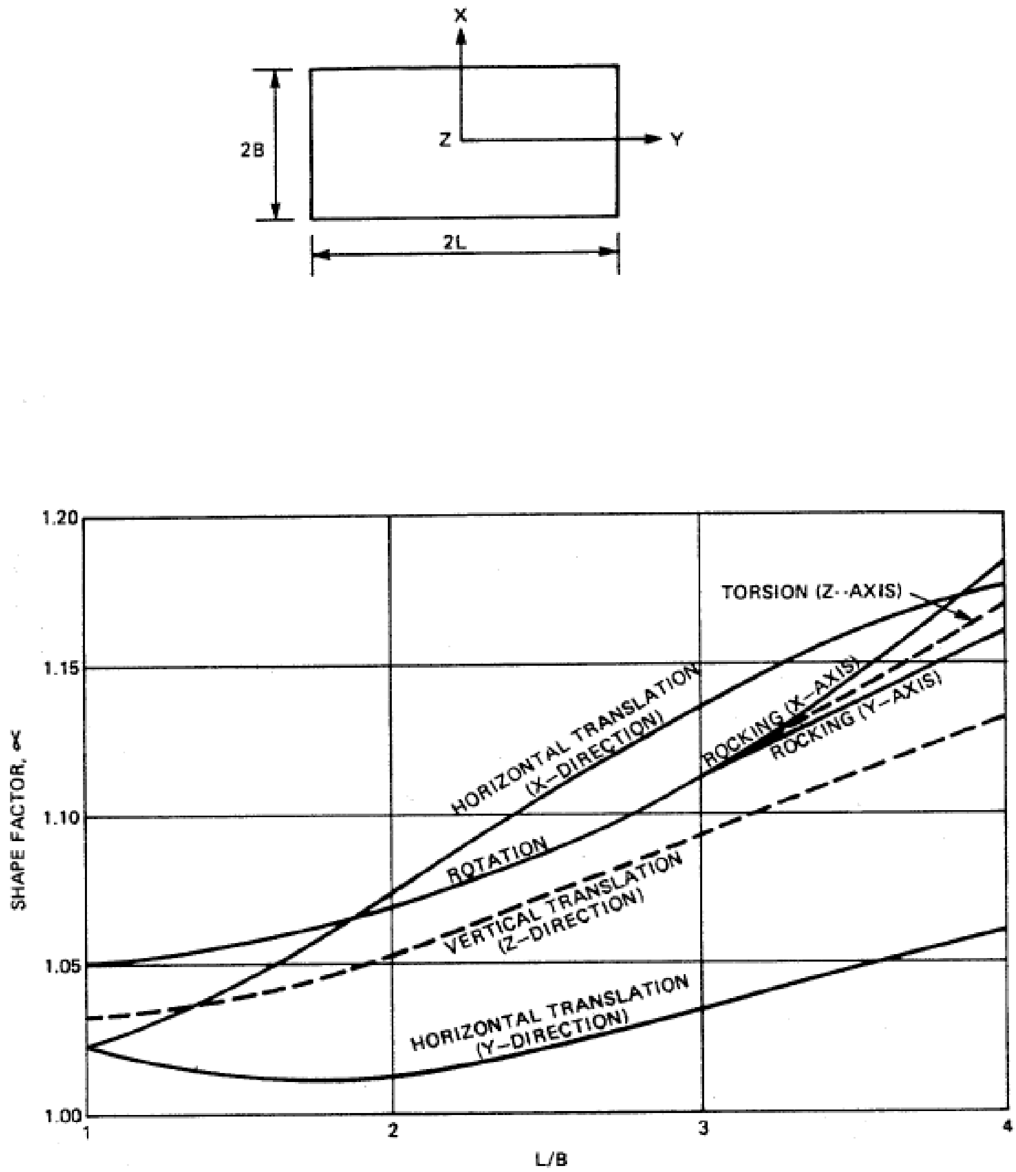
EQUIVALENT RADIUS:

TRANSLATIONAL: $$R_o = \sqrt{\frac{4BL}{\pi}}$$

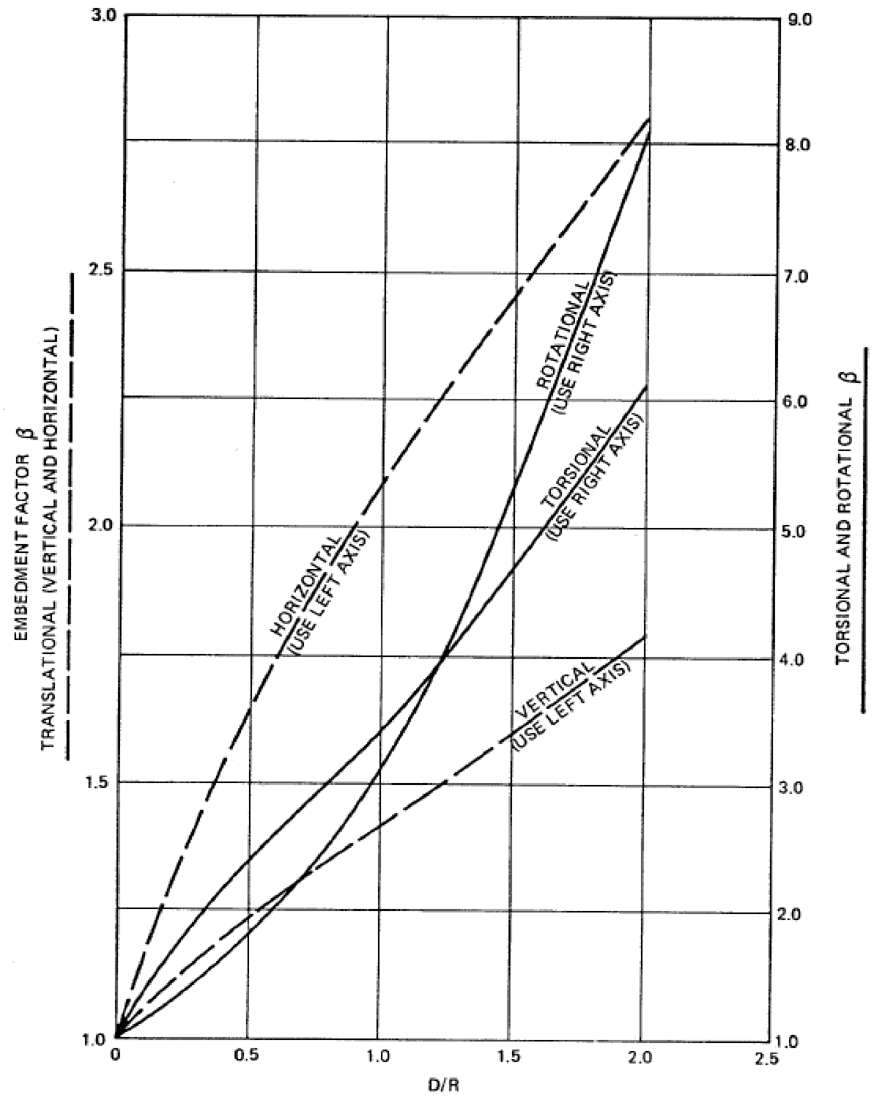
ROTATIONAL: $$R_1 = \left[\frac{4BL\,(4B^2 + 4L^2)}{6\pi}\right]^{1/4} \quad \text{(x-AXIS ROCKING)}$$

$$R_2 = \left[\frac{(2B)^3\,(2L)}{3\pi}\right]^{1/4} \quad \text{(y-AXIS ROCKING)}$$

$$R_3 = \left[\frac{(2B)\,(2L)^3}{3\pi}\right]^{1/4} \quad \text{(z-AXIS TORSION)}$$



Shape Factor for Rectangular Footings
Figure 7-24



Embedment Factor
Figure 7-25

7.3 Column Design

7.3.1 Preliminary Plan Stage

The preliminary plan stage determines the initial column size, column spacing, and bridge span length based on a preliminary analysis. Columns are spaced to give maximum structural benefit except where aesthetic considerations dictate otherwise. Piers normally are spaced to meet the geometric and aesthetic requirements of the site and to give maximum economy for the total structure. Good preliminary engineering judgment results in maximum economy for the total structure.

The designer may make changes after the preliminary plan stage. The supervisor will review all changes and will determine if the changes need to be reviewed by the Bridge Projects Engineer or the State Bridge and Structures Architect.

Tall piers spaced farther apart aesthetically justify longer spans. Difficult and expensive foundation conditions will also justify longer spans. Span lengths may change in the design stage if substantial structural improvement and/or cost savings can be realized. The designer should discuss the possibilities of span lengths or skew with the supervisor as soon as possible. Changes in pier spacing at this stage can have significant negative impacts to the geotechnical investigation.

Column spacing should minimize column dead load moments. Multiple columns are better suited for handling lateral loads due to wind and/or earthquake. The designer may alter column size or spacing for structural reasons or change from a single-column pier to a multicolumn pier.

7.3.2 General Column Criteria

Columns should be designed so that construction is as simple and repetitious as possible. The diameter of circular columns should be a multiple of one foot. Rectangular sections should have lengths and widths that are multiples of 3 inches. Long rectangular columns are often tapered to reduce the amount of column reinforcement required for strength. Tapers should be linear for ease of construction.

Understanding the effects on long columns due to applied loads is fundamental in their design. Loads applied to the columns consist of reactions from loads applied to the superstructure and loads applied directly to the columns. For long columns, it may be advantageous to reduce the amount of reinforcement as the applied loads decrease along the column. In these cases, load combinations need to be generated at the locations where the reinforcement is reduced.

Construction Joints

Bridge Plans will show column construction joints at the top of footing or pedestal and at the bottom of crossbeam. Optional construction joints with roughened surfaces should be provided at approximately 30-foot vertical spacing.

Modes of Failure

A column subject to axial load and moment can fail in several modes. A “short” column can fail due to crushing of the concrete or to failure of the tensile reinforcement. A “long” column can fail due to elastic buckling even though, in the initial stages, stresses are well within the normal allowable range. Long column failure is normally a combination of stability and strength failure that might occur in the following sequence:

1 Axial load is applied to the column.
2. Bending moments are applied to the column, causing an eccentric deflection.
3. Axial loads act eccentrically to the new column center line producing P-Δ moments which add directly to the applied moments.

4. P-Δ moments increase the deflection of the column and lead to more eccentricity and moments.
5. The P-Δ analysis must prove the column loading and deflection converges to a state where column stresses are acceptable. Otherwise, the column is not stable and failure can be catastrophic.

Bridge vs. Building Columns

Unlike building columns, bridge columns are required to resist lateral loads through bending and shear. As a result, these columns may be required to resist relatively large applied moments while carrying nominal axial loads. In addition, columns are often shaped for appearance. This results in complicating the analysis problem with non-prismatic sections.

7.3.3 Column Design Flow Chart

The Figure 7-26 illustrates the basic steps in the column design process.

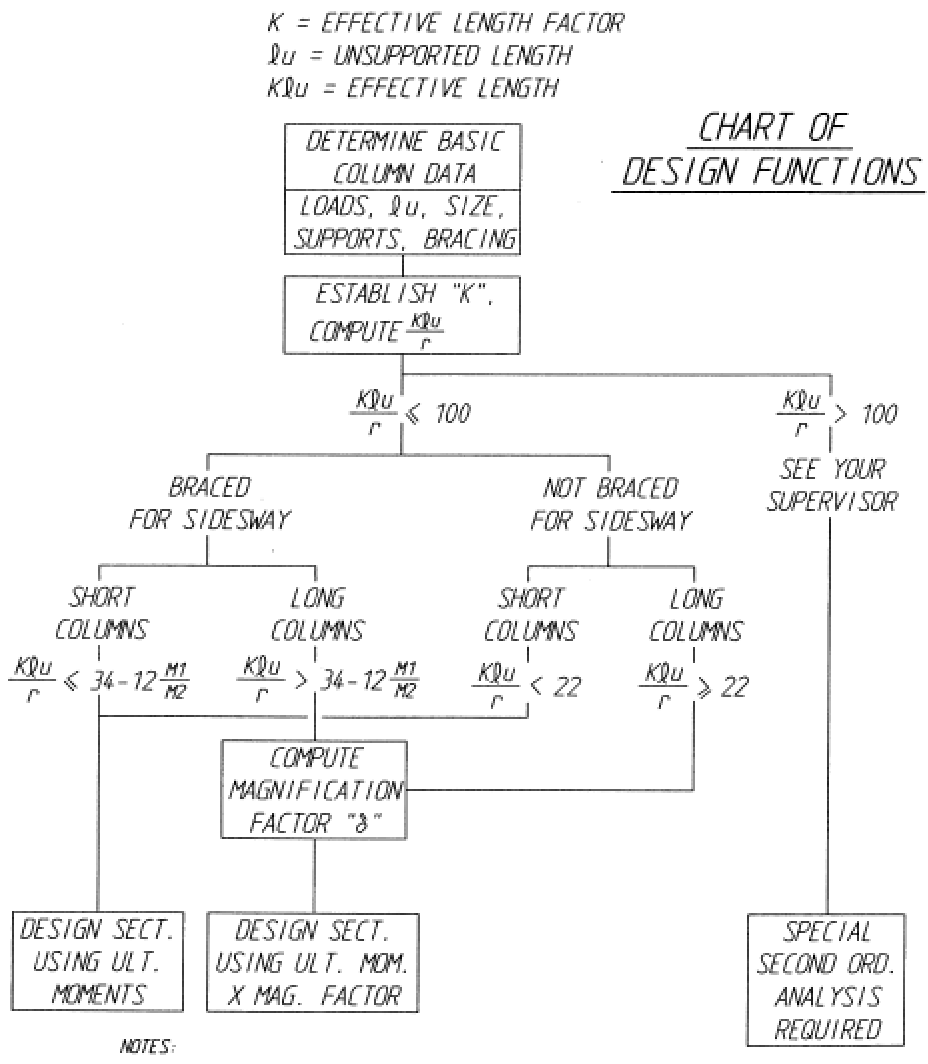


Figure 7-26

7.3.4 *Slenderness Effects*

This BDM section supplements and clarifies AASHTO specifications. The goal of a slenderness analysis is to estimate the additional bending moments in the columns that are developed due to axial loads acting upon a deflected structure. Two primary analysis methods exist: the moment magnifier method and the second-order analysis. The designer must decide which method to use based upon the slenderness ratio (kL_u/r) of the column(s).

Method 1: Allowed if $kL_u/r <= 100$. BDM Section 7.3.3.2 discusses the approximate moment magnifier method that is generally more conservative and easier to apply.

Method 2: Recommended by AASHTO for all situations and is mandatory for $kL_u/r > 100$. BDM Section 7.3.6 discusses a second-order structural analysis that accounts directly for the axial forces and can lead to significant economy in the final structure.

In general, tall thin columns and piles above ground are considered unbraced and larger short columns and pilings below grade are considered braced.

Braced or Unbraced Columns

In a member with loads applied at the joints, any significant deflection "side ways" indicated the member is unbraced. The usual practice is to consider the pier columns as unbraced in the transverse direction. The superstructure engages girder stops at the abutment and resists lateral sidesway due to axial loads. However, pier lateral deflections are significant and are considered unbraced. Short spanned bridges may be an exception.

Most bridge designs provide longitudinal expansion bearings at the end piers. Intermediate columns are considered unbraced because they must resist the longitudinal loading. The only time a column is braced in the longitudinal direction is when a framed bracing member does not let the column displace more than L/1500. L is the total column length. In this case, the bracing member must be designed to take all of the horizontal forces.

7.3.5 *Moment Magnification Method*

The moment magnification method is described in AASHTO LRFD Section 4.5.3.2.2. The following information is required.

- Column geometry and properties: E, I, L_u, and k.
- All Strength or Extreme Event loads obtained from conventional elastic analyses using appropriate stiffness and fixity assumptions and column under strength factor (ϕ).

Computations of effective length factors, k, and buckling loads, P_c, are not required for a second-order analysis, though they may be helpful in establishing the need for such an analysis. In general, if magnification factors computed using the AASHTO Specifications are found to exceed about 1.4, then a second-order analysis may yield substantial benefits.

7.3.6 *Second-Order Analysis*

A second-order analysis includes the influence of axial loads on the deflected structure is required under certain circumstances, and may be advisable in others. It can lead to substantial economy in the final design of many structures. The designer should consider should discuss the situation with the supervisor before proceeding with the analysis. The ACI Building Code (ACI 318 R-02, section 10.13.4.1) should be consulted when carrying out a second-order analysis.

For columns framed together, the entire frame should be analyzed as a unit. Analyzing individual columns result in overly conservative designs for some columns and non-conservative results for others. This is a result of redistribution of the lateral loads in response to the reduced stiffness of the compression members. For example, in a bridge with long, flexible columns and with short, stiff columns both integrally connected to a continuous superstructure, the stiff columns will tend to take a larger proportion of the lateral loading as additional sidesway under axial loads occurs.

A. Design Methods for a Second-Order Analysis

The preferred method for performing a second-order analysis of an entire frame or isolated single columns is uses a nonlinear finite element program, such as GTSTRUDL, with appropriate stiffness and restraint assumptions. The factored group loads are applied to the frame, including the self-weight of the columns. The model is then analyzed using the nonlinear option available in GTSTRUDL. The final design moments are obtained directly from the analysis.

P-Δ moments are added to the applied moments using an iterative process until stability is reached. The deflections should converge within 5% of the total deflection. Analysis must include the effect of the column weight; therefore, the axial dead load must be adjusted as follows:

$$P_u = P_u + {}^{1}/{}_{3} \text{ (factored column weight).}$$

B. Applying Factored Loads

For a second-order analysis, loads are applied to the structure and the analysis results in member forces and deflections. It must be recognized that a second-order analysis is non-linear and the commonly assumed principle of superposition may not be applicable. The loads applied to the structure should be the entire set of factored loads for the load group under consideration. The analysis must be repeated for each group load of interest. The problem is complicated by the fact that it is often difficult to predict in advance which load groups will govern.

For certain loadings, column moments are sensitive to the stiffness assumptions used in the analysis. For example, loads developed as a result of thermal deformations within a structure may change significantly with changes in column, beam, and foundation stiffness. Accordingly, upper and lower bounds on the stiffness should be determined and the analysis repeated using both sets to verify the governing load has been identified.

C. Member Properties

As with a conventional linear elastic frame analysis, various assumptions and simplifications must be made concerning member stiffness, connectivity, and foundation restraint. Care must be taken to use conservative values for the slenderness analysis. Reinforcement, cracking, load duration, and their variation along the members are difficult to model while foundation restraint will be modeled using soil springs.

7.4 Column Reinforcement

7.4.1 Minimum Longitudinal Reinforcement Ratio

The reinforcement ratio is the steel area divided by the gross area of the section - A_s/A_g. The maximum reinforcement ratio shall be 0.06. The minimum reinforcement ratio shall be 0.01. The ratio may be reduced to 0.005 if all loads can be carried on a reduced section with similar shape and reinforcement ratio is equal to or greater than 0.01. The column dimensions are reduced by the same ratio to obtain the similar shape. The reduced section properties are not used for design or modeling.

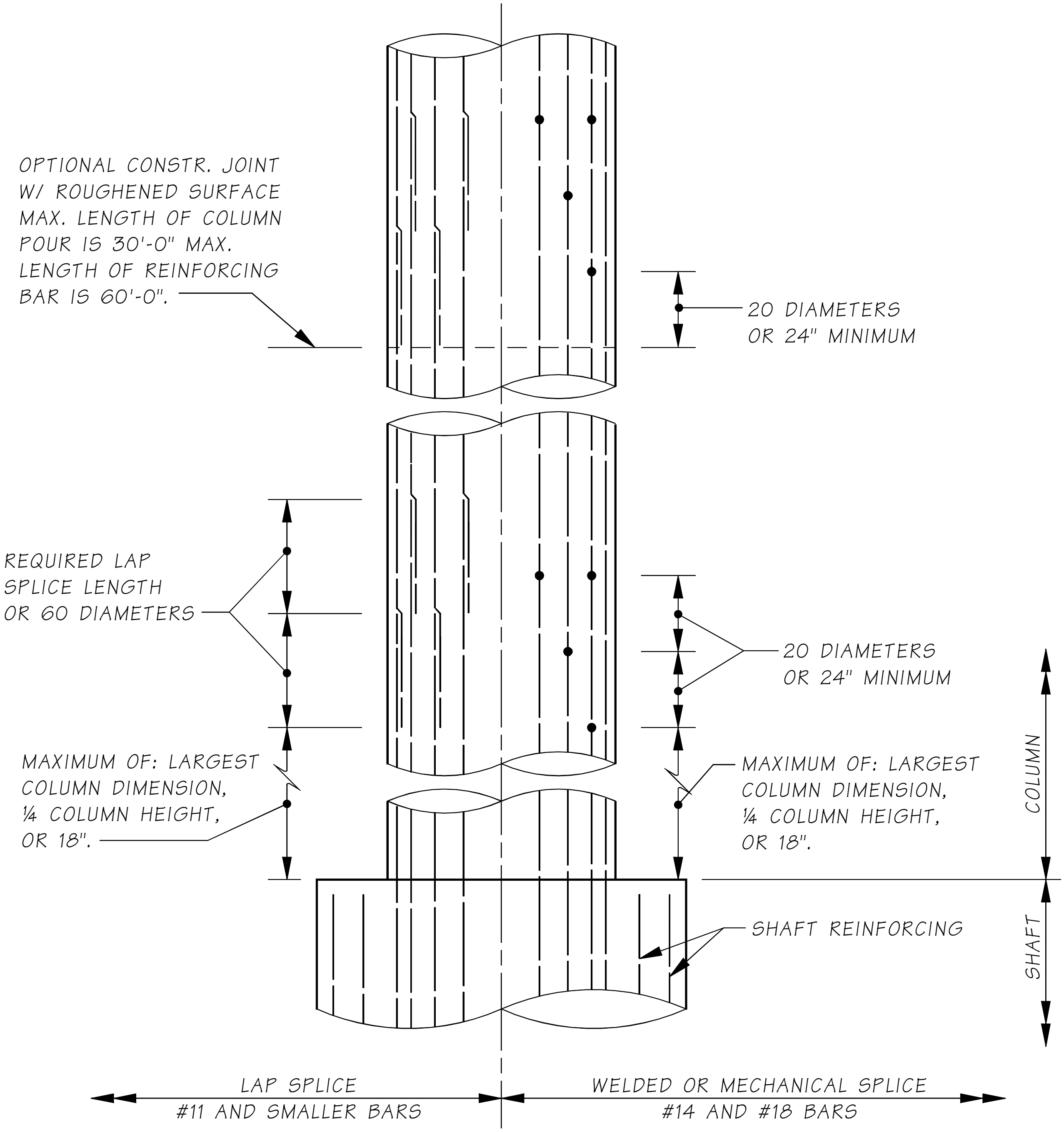
7.4.2 Longitudinal Splices

Column reinforcement shall not be spliced at points of maximum moment, plastic hinge locations, or in columns less than 30 feet long between the top of footing and the bottom of crossbeam. The Bridge Plans must show splice location, length, and optional weld details. The Standard Specifications cover approved mechanical splices.

All splices of No. 7 and larger bars shall be staggered. For column intermediate construction joints, the shortest staggered lap bar shall project above the joint 60 bar diameters or 20 bar diameters for welded splices. If plans indicate the splice is OPTIONAL, the Special Provisions will define the method of payment for the splice steel. Figure 7-27 shows the standard practice for staggered lap splice locations.

Longitudinal # 11 and smaller bars will use lap slices. When space is limited, #11 and smaller bars can use welded splices, an approved mechanical butt splice, or the top bar can be bent inward (deformed by double bending) to lie inside and parallel to the bars below. When the bar size exceeds #11, a welded splice or an approved mechanical butt splice is required. The smaller bars in the splice determine the type of splice required.

AASHTO requires splices to fall within the middle one-half of the column. In extremely tall columns where a 60-foot bar cannot reach the middle half, splices should not be closer than 30 feet from the columns ends.



Column Lap Splice Locations
Figure 7-27

7.4.3 Ties and Spirals

Square stirrup ties or spirals are required in all columns to resist shear forces and to maintain the column structural integrity after catastrophic forces have cracked or removed the cover. Plans should show two section views of transverse reinforcing differentiating the column ends and the typical middle sections.

Confinement Zones

The column end section will only be used for the confinement zones, where it must both provide confinement and resist shear. Spirals and cross-ties in the confinement zones are limited in size to #6 bars. Shear reinforcement can be made up of several reinforcing elements with 135° seismic hooks extending into the core. Cross-ties have a 180° hook on one end and a 90° hook on the other. The hooks should alternate both horizontally and vertically. The tie must engage the peripheral spiral or stirrup and be tied to the longitudinal reinforcement. The designer should check to make sure the 180° hook can fit between the spiral and the longitudinal bars.

In regions between confinement zones, where confinement is not required, the transverse reinforcing needs to resist the column shear.

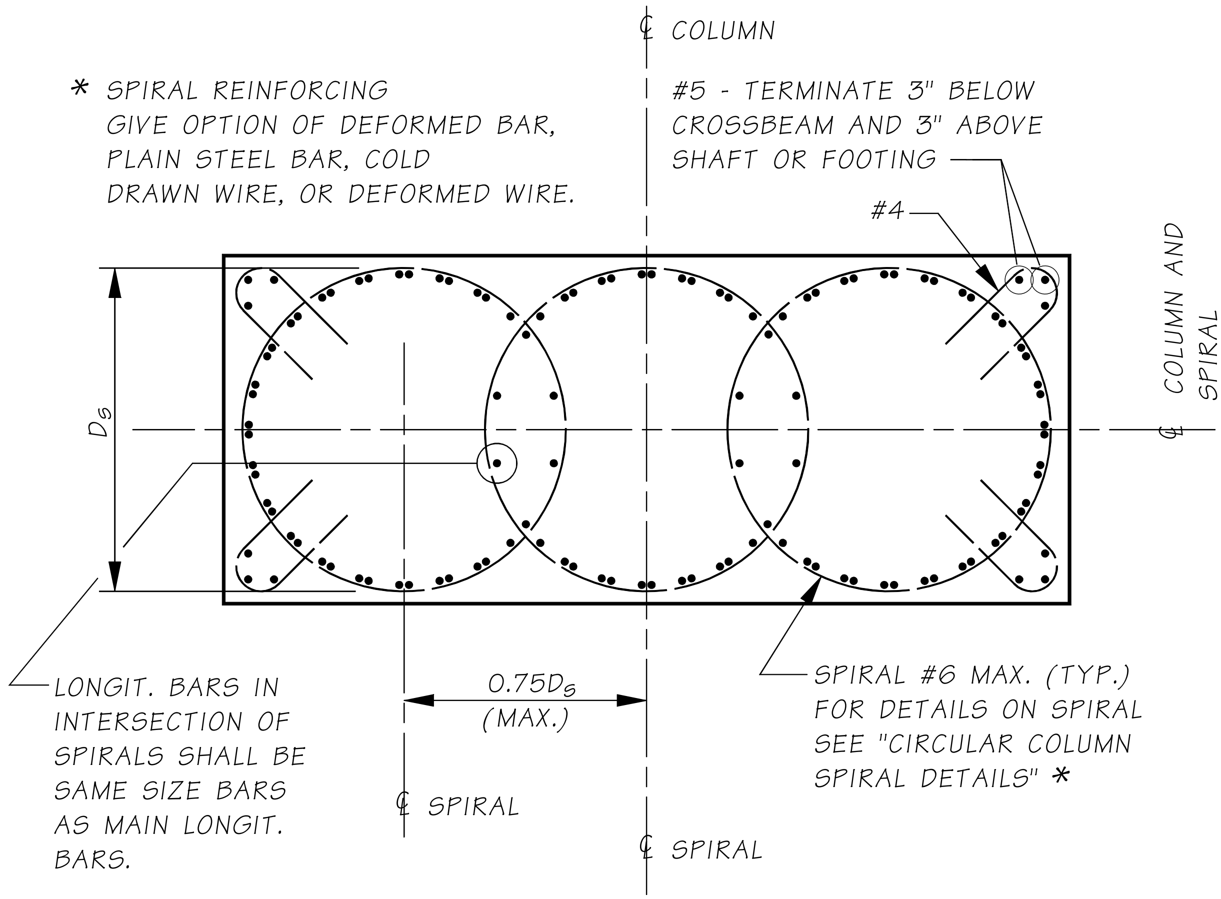
AASHTO Shear Requirements

The area of the transverse reinforcement required to resist the column shear is defined in Article "Column Shear and Transverse Reinforcement" of the Guide Specifications and AASHTO Article "Shear." The area of transverse reinforcement required for confinement is determined from Guide Specifications Article "Spacing of Transverse Reinforcement for Confinement" for spirals and ties. The area of transverse reinforcing in the confinement zones is the larger of the two requirements. Transverse reinforcement may be provided by spirals, hoops, or cross-ties.

Spiral Reinforcement

All circular columns are to use spirals. Standard sizes for column spiral use are #4 or #5 deformed bar, 1/2-inch diameter or 5/8-inch diameter plain steel bar, or W20 or W31 cold drawn wire. Plans should state all three spiral options. For example, "# 4, 1/2 inch diameter, or W20 spiral". To facilitate concrete placement, the minimum spiral pitch is 1 inch or 1 1/3 times the maximum coarse aggregate size. Terminate column spirals at the top and the bottom with a seismic hook. Specify spiral weight in the bar list based on the full height. The fabricator will provide required splices.

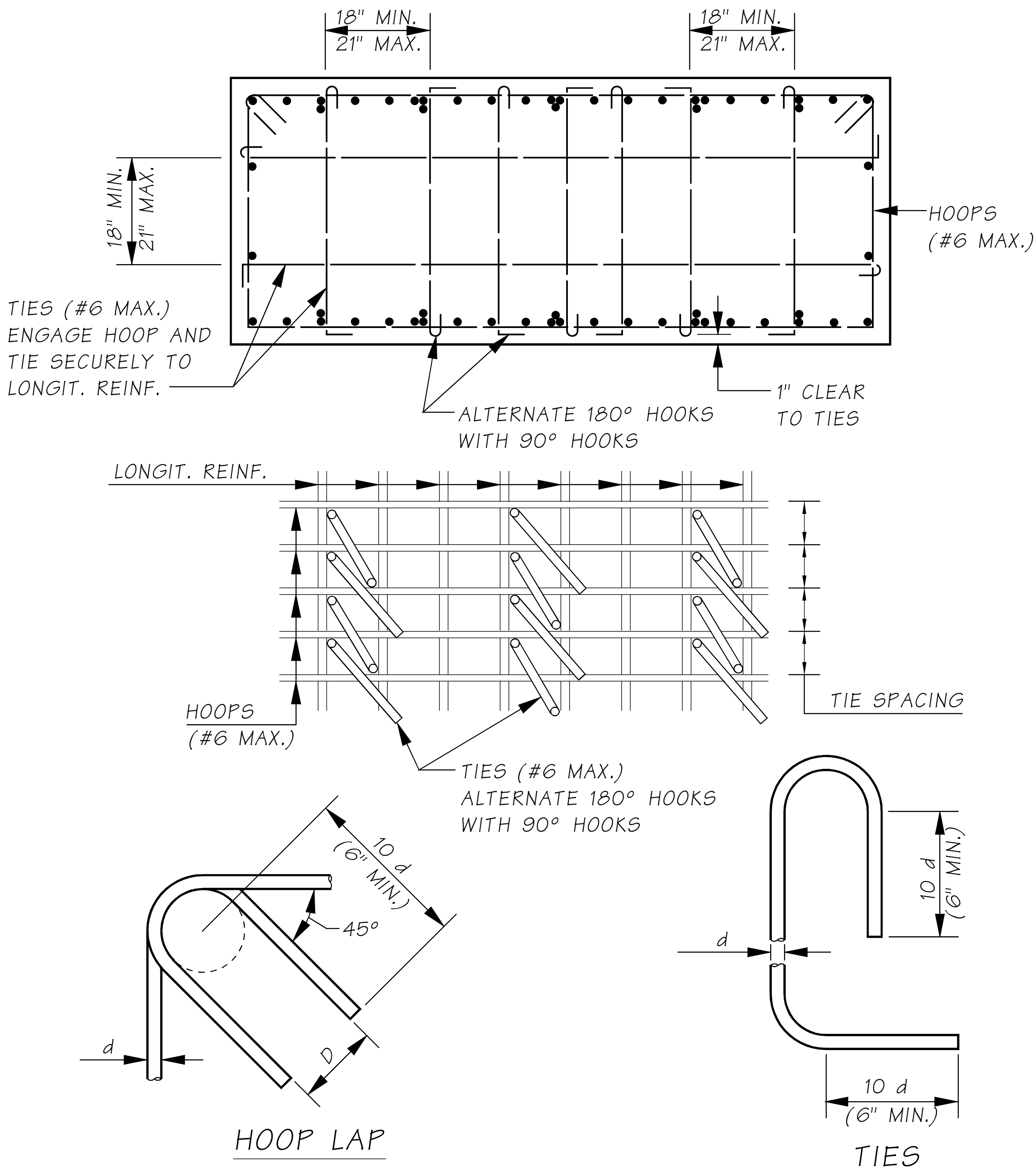
A rectangular column with constant width or vertical taper and spiral reinforcement is shown in Figure 7-28. The same provisions as a spirally reinforced circular column apply.



DEFORMED BAR	PLAIN STEEL BAR	COLD DRAWN WIRE	DEFORMED WIRE
#4	½"ø	W20	D20
#5	⅝"ø	W31	D31
#6	¾"ø	W44	D44

Constant and Tapered Rectangular Column Section
Seismic Zones C and D
Figure 7-28

A rectangular or tapered column with stirrups and cross-ties is shown in Figures 7-29. The maximum vertical stirrup or cross-tie spacing in confinement zones and at lap splices is 4 inches for Seismic Performance Categories C and D and 6 inches for Seismic Performance Categories A and B. The minimum vertical spacing is $2^1/_2$ inches for concrete placement. Stirrup spacing layout and detailing should be based on longitudinal lap locations.



Constant and Tapered Rectangular Column Ties
Seismic Zones A and B
Figure 7-29

7.4.4 Longitudinal Development and Confinement Steel

Columns framed a footing, crossbeam, or frame must have confinement. The confinement reinforcement lengths are the same for columns of any shape. The confinement in the horizontal members starts at the end of the straight longitudinal column bars. For hooked bars, confinement starts at the beginning of the longitudinal column bar curvature. Confinement reinforcing extends into the column the maximum of the following three criteria. Extension is measured from the bottom of crossbeam or top of footing or pedestal.

1) 1/6 the clear column height

2) Maximum horizontal column dimension – Note: for wall type piers where plastic hinging occurs only along the weak axis, use the short dimension.

3) 18 inches

Crossbeams

All column longitudinal rebar shall terminate at least 3” below the top longitudinal crossbeam reinforcement. The designer shall ensure that the development length of the column longitudinal rebar is at least equal to 1.25 L_d. This development length may be modified by the applicable modification factors from AASHTO LRFD section 5.11.2.

The top of columns in crossbeam should avoid hooks. If the crossbeam is not deep enough to develop straight bars, 180° hooks generally provide less congestion. For columns on drilled shafts, hooked layouts must allow enough room for the contractor to remove casing that is the diameter of the shaft. A detail showing horizontal lower crossbeam rebar and vertical column rebar is preferred but not required.

Footings

Longitudinal reinforcement at the bottom of a column should extend into the footing and rest on the bottom mat of footing reinforcement with standard 90° hooks. Embedment must be at least 1.25 1_{dh} (1_{dh} is development length of a standard hook). This development length may be further modified by the applicable modification factors from AASHTO LRFD Section 5.11.2.

Drilled Shafts

Longitudinal column bars in drilled shafts are typically straight. For extension of column longitudinal bars into shafts, see Figure 7-30.

Confinement Layout and Longitudinal Length
Figure 7-30

7.4.5 Column Hinges

The area of the hinge bars in square inches is as follows:

$$As = \frac{\frac{(P_u)}{2} + \left[\frac{P_u^2}{4} + V_u^2\right]^{1/2}}{0.85\, F_y \operatorname{Cos} \theta}$$

Where:

P_u is the factored axial load

V_u is the factored shear load

F_y is the reinforcing yield strength (60 ksi)

θ is the angle of the hinge bar to the vertical

The development length required for the hinge bars is 1.25 times that described in AASHTO Article "Development of Flexural Reinforcement." Figure 7-32 shows some typical hinge details. Space the ties and spirals to satisfy Article "Spacing of Transverse Reinforcement for Confinement" of the Guide Specifications, AASHTO Article "Shear," or a maximum of 12 inches (6 inches if longitudinal bars are bundled). Premolded joint filler should be used to assure the required rotational capacity. There should also be a shear key at the hinge bar location.

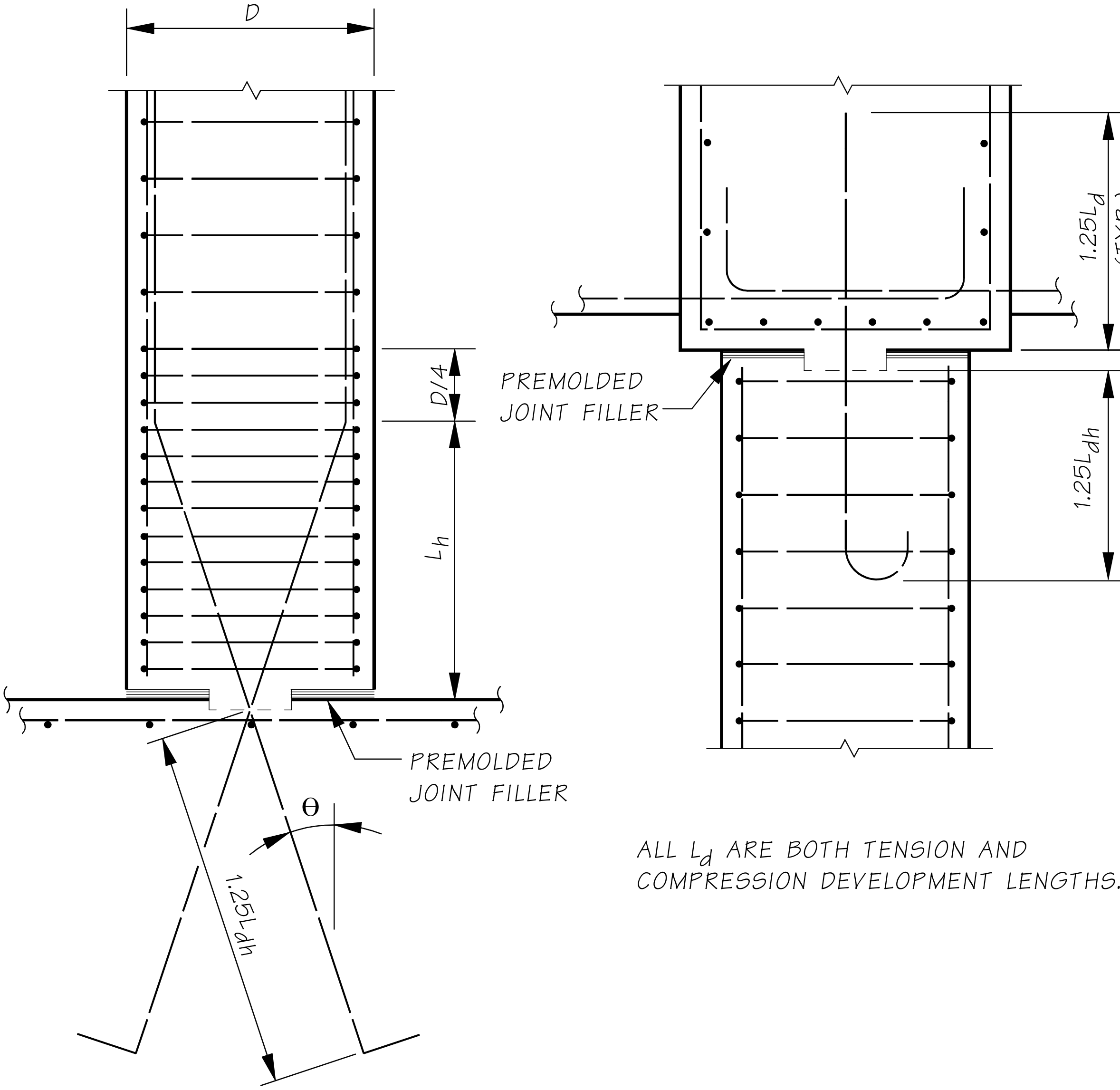
When the hinge reinforcement is bent, additional confinement reinforcing may be necessary to take the horizontal component from the bent hinge bars. The maximum spacing of confinement reinforcing for the hinge is the smaller of that required above and the following:

$$S_{max} = \frac{A_v F_y}{\left[\frac{P_u \operatorname{Tan} \theta}{0.85\, 1_h} + \frac{V_s}{d}\right]}$$

Where:

A_v, V_s, and d are as defined in AASHTO Article "Notations" and 1_h is the distance from the hinge to where the bend begins.

Continue this spacing one-quarter of the column width (in the plane perpendicular to the hinge) past the bend in the hinge bars.



Hinge Details
Figure 7-31

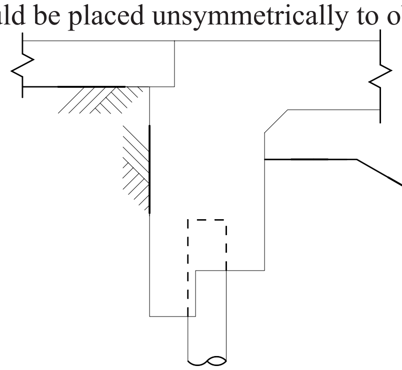
7.5 Abutment Design and Details

7.5.1 Abutment Types

There are five abutment types described in the following section that have been used by the Bridge and Structures Division. The representative types are intended for guidance only and may be varied to suit the requirements of the bridge being designed.

Pile Cap Abutments

Earth pressures on some pile caps are either negligible or very small (when the lateral force on each pile is less than 6 kips), and vertical dead load and live load are the major effects. The design of this type of abutment is like that of a crossbeam, and transverse bending as well as shear shall be investigated for the spans between the piles. For the analysis of the pile cap, the wheel loads should be placed for the maximum moment on the pile cap. For the analysis of the piles, the wheel loads should be placed unsymmetrically to obtain the largest pile reaction.

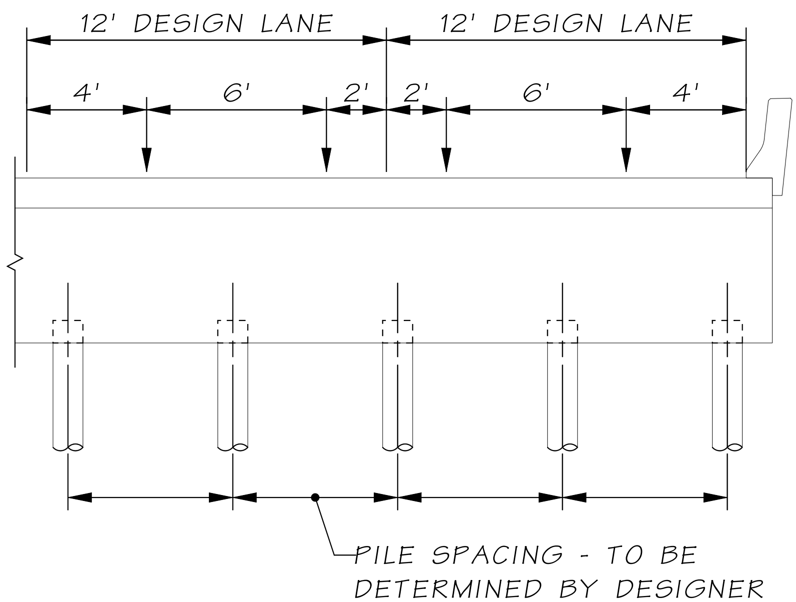


Pile Cap Abutment
Figure 7-32

For narrow bridges (one-lane ramps and two-lane bridges without skew) the transverse live load moment on the abutment shall be taken about the center of gravity of the pile group assuming the abutment to be a rigid beam. The maximum pile reaction from transverse effect will then be $P/N + M_t/S$, where P is the total vertical load, N is the total number of piles, M_t is the transverse moment about the centerline of abutment and S is the transverse pile modulus. This analysis is only valid if the lateral forces from earth pressure, etc. are less than 6 kips per pile and all the piles have no batter.

For wide bridges (2 lanes with skew and wider) the abutment may be assumed to act as a flexible beam on knife-edge supports. The maximum pile live load reaction from transverse loading can be obtained by assuming the abutment acts as a simple beam between piles and each wheel load (in the design lane or approach lane) is proportionally distributed to the adjacent piles (see Figure 7-33). Transverse moments and shears may be found assuming the spans between piles as semi-simply supported: i.e. maximum positive or negative moment = 0.80 times the simple beam moment. Maximum shear = simple beam shear. This analysis is valid for piles with a stiffness much less than the pile cap.

For pile caps with lateral loads greater than 6 kips, with battered piles, or for piles with a stiffness about the same magnitude as the pile cap, such as shafts, the analysis for the pile cap should be as a crossbeam, The analysis for the piles should include the lateral capacity of the pile.



Lane Placement for Maximum Live Load Reaction (Center Pile)

Figure 7-33

Stub Abutments

Stub abutments are short abutments where the distance from the girder seat to top of footing is less than approximately 4 feet, see Figure 7-34. The footing and wall can be considered as a continuous inverted T-beam. The analysis of this type abutment shall include investigation into both bending and shear stresses parallel to centerline of bearing. If the superstructure is relatively deep, earth pressure combined with longitudinal forces from the superstructure may become significant

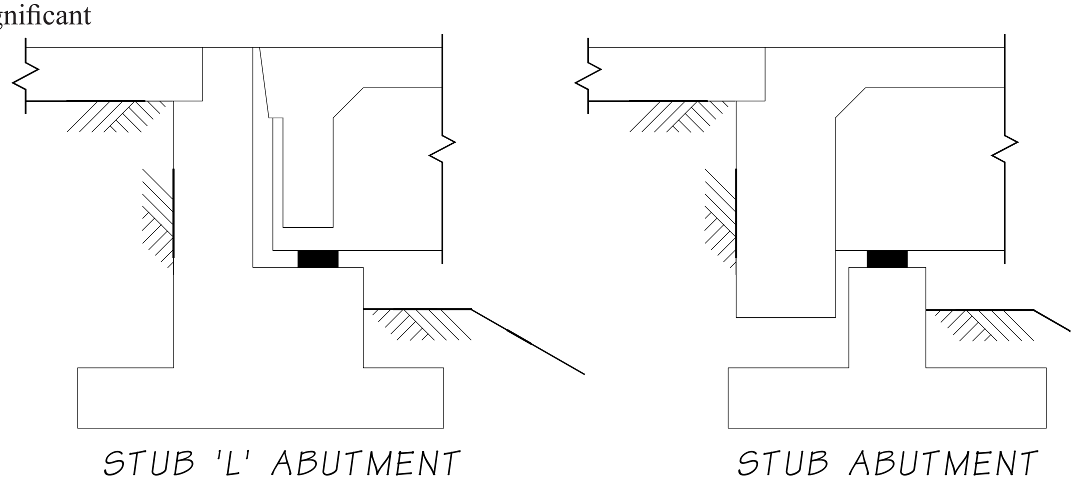


Figure 7-34

Cantilever Abutments

If the height of the wall from the bearing seat down to the bottom of the footing exceeds the clear distance between the girder bearings, the assumed 45° lines of influence from the girder reactions will overlap, and the dead load and live load from the superstructure can be assumed equally distributed over the abutment width. The design may then be carried out on a per foot basis as described in the BDM Section 7.5.9. The primary structural action takes place normal to the abutment, and the bending moment effect parallel to the abutment may be neglected in most cases. The wall is assumed to be a cantilever member fixed at the top of the footing and subjected to axial, shear, and bending loads.

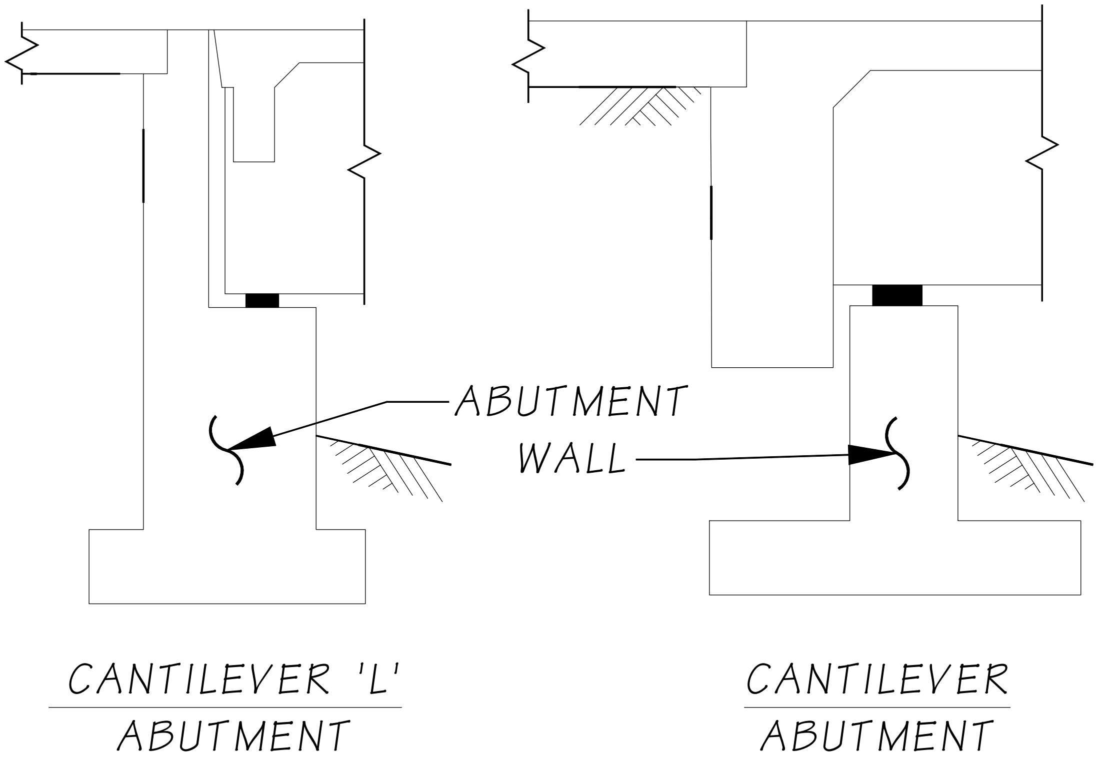
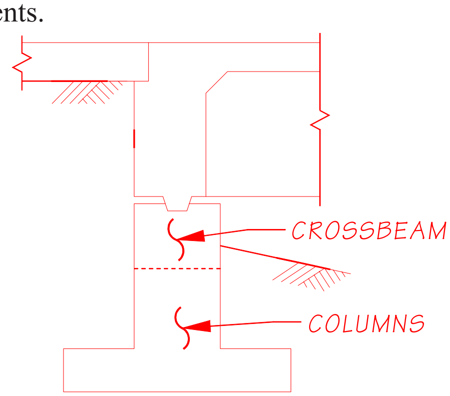


Figure 7-35

Spill-Through Abutments

The analysis of this type of abutment is similar to that of an intermediate pier,see Figure 7-36. The crossbeam shall be investigated for vertical loading as well as earth pressure and longitudinal effects transmitted from the superstructure. Columns shall be investigated for vertical loads combined with horizontal forces acting transversely and longitudinally. For earth pressure acting on rectangular columns, assume an effective column width equal to 1.5 times the actual column width. Short, stiff columns may require a hinge at the top or bottom to relieve excessive longitudinal moments.

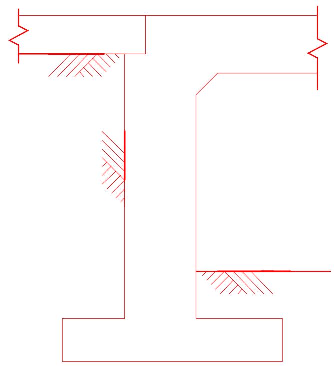


Spill-Through Abutment

Figure 7-36

Rigid Frame Abutments

Abutments that are part of a rigid frame are generically shown in Figure 7.5.1-6. At-Rest earth pressures (EH) will apply to these structures. The abutment design should include the live load impact factor from the superstructure. However, impact shall not be included in the footing design. The rigid frame itself should be considered restrained against sidesway for live load only. AASHTO Chapter 12 addresses loading and analysis of rigid frames that are buried (box culverts).



Rigid Frame Abutment
Figure 7-37

7.5.2 Embankment at Abutments

The minimum clearances for the embankment at the front face of abutments shall be as indicated on Standard Plan Sheet H-9. At the ends of the abutment, the fill may be contained with wing walls or in the case of concrete structures, placed against the exterior girders.

7.5.3 Abutment Loading

In general, bridge abutment loading will be in accordance with AASHTO LRFD Chapter 3. The following simplifications and assumptions may be applied to the abutment design. See Section 7.5 for a force diagram of typical loads as they are applied to an abutment spread footing.

Dead load - DC

Approach slab dead load reaction taken as 2 kips/ foot of wall applied at the pavement seat.

Active earth pressure (EH) and unit weight of backfill and toe fill (EV) will be provided in a Geotechnical Report. The toe fill should be included in the analysis for overturning if it adds to overturning.

The passive earth pressure exerted by the fill in front of the abutment is usually neglected in the design. The Geotechnical Division should be contacted to determine if passive resistance might be considered for analysis of sliding stability. Passive resistance in front of footing is not dependable due to potential for erosion, scour, or future excavation in front of footing.

Live load - LL

Live load impact does not apply to the abutment. Approach slab live load reaction (without IM) applied at the pavement seat may be assumed to be 4.5 kips per foot of wall for HL-93 loading, see BDM Section 10.5 for approach slab design assumptions. Abutment footing live loads may be reduced (by approximately one axle) if one design truck is placed at the bridge abutment with an approach span. Adding the pavement seat reaction to the bearing reaction duplicates the axle load from two different design truck configurations.

If approach slabs are not provided in the bridge plans, a live load surcharge (LS) applies.

Earthquake Load - EQ

Superstructure loads will be transmitted to the substructure through bearings, girder stops or restrainers. As an alternate, the superstructure may be rigidly attached substructure.

The horizontal earth pressure load (EQ_{soil}) will be the Mononobe-Okabe (M-O)active pressure coefficient, as described in the LRFD Chapter 11, Appendix 11.1.1.1. This applies M-O as a uniform pressure to the wall with the resultant force located at 0.5H. For more information on Mononobe-Okabe and AASHTO application, see GDM Section 15.4.2.9.

Footing supported walls and abutments that are free to translate or move during a seismic event shall use Mononobe-Okabe soil pressure. The vertical acceleration, k_v, shall be set equal to 0. This also applies to portions substructure isolated from the superstructure by bearings.

Pile or shaft walls and abutments that are not free to translate or move during a seismic event shall use a horizontal acceleration of 1.5 times peak ground acceleration. The vertical acceleration shall be set equal to 0. See GDM Section 15.4.2.7 for descriptions of flexible and non-yielding walls.

Seismic inertial force of the substructure (EQ_{abut}) is the horizontal acceleration coefficient times the weight of the abutment (including footing). This force acts horizontally in the same direction as the earth pressure, at the mass centroid of the abutment. Seismic inertia force is only applied for stability and sliding analysis. EQ_{abut} shall not be used to determine the reinforcement required in the abutment.

Bearing Forces – TG Strength and Extreme Event II

For strength design, the bearing shear forces should be based on ½ of the seasonal temperature change. This force is applied in the direction that causes the worst case loading.

For extreme event II, calculate the maximum friction force (when the bearing slips) and apply in the direction that causes the worst case loading.

7.5.4 WSDOT Temporary Construction Load Cases

Case 1: Superstructure Built after Backfill at Abutment

If the superstructure is to be built after the backfill is placed at the abutments, the resulting temporary loading would be the maximum horizontal force with the minimum vertical force. During the abutment design, a load case shall be considered to check the stability and sliding of abutments after placing backfill but prior to superstructure placement. This load case is intended as a check for a temporary construction stage, and not meant to be a controlling load case that would govern the final design of the abutment and footing. This loading will generally determine the tensile reinforcement in the top of the footing heel.

If this load case check is found to be satisfactory, a note will be added to the general notes in the contract plans and the contactor will not be required to make a submittal requesting approval for early backfill placement. This load case will include a 2-foot deep soil surcharge for the backfill placement equipment (LS) as covered by the WSDOT Standard Specifications article 2-03.3(14)I.

Case 2: Wingwall Overturning

It is usually advantageous in sizing the footing to release the false work from under the wing walls after some portion of the superstructure load is applied to the abutment. A note can cover this item, when applicable, in the sequence of construction on the plans.

7.5.5 Abutment Bearings and Girder Stops

All structures shall be provided with some means of restraint against lateral displacement at the abutments due to earthquake, temperature and shrinkage, wind, earth pressure, etc. Such restraints may be in the form of concrete hinges, concrete girder stops with or without vertical elastomeric pads, or pintles in metal bearings. Other solutions are possible. Article “Connection Design Forces” of the Guide Specifications for Seismic Design of Highway Bridges describe longitudinal linkage force and hold-down devices required.

All prestressed girder bridges in Western Washington (within and west of the Cascade mountain range) shall have girder stops between all girders at abutments and intermediate piers. This policy is based on fact that the February 28, 2001 Nisqually earthquake caused significant damage to girder stops at bridges where girder stops were not provided between all girders. In cases where girder stops were cast prior to placement of girders and the 3" grout was placed after setting the girders, the 3" grout pads were severely damaged and were displaced from their original position.

Abutment Bearings

The longitudinal forces from the superstructure are normally transferred to the abutments through the bearings. The calculated longitudinal movement shall be used to determine the shear force developed by the bearing pads at the abutments. The Modulus of Elasticity of Neoprene at 70°F (21°C) shall be used for determining the shear force. However, the force transmitted through a bearing pad shall be limited to that which causes the bearing pad to slip. Normally, the maximum load transferred through a teflon sliding bearing is 6 percent and through an elastomeric bearing pad is 20 percent of the dead load reaction of the superstructure. For Extreme Event I, assume the end diaphragm is in contact with abutment wall and no load transfer through the bearings. The bearing force shall not be added to seismic earth pressure forces.

When the force transmitted through the bearing pads is very large, the designer should consider increasing the bearing pad thickness, using TFE sliding bearings and/or utilizing the flexibility of the abutment as a means of reducing the horizontal design force. When the flexibility of the abutment is considered, it is intended that a simple approximation of the abutment deformation be made.

Bearing Seats

The bearing seats shall be wide enough to accommodate the size of the bearings used with a minimum edge dimension of 3 in. and satisfy the requirements of LRFD Section 4.7.4.4. On L-abutments, the bearing seat should be sloped away from the bearings to prevent a build up or pocket of water at the bearings. The superelevation and profile grade of the structure should be considered for drainage protection. Normally, a $^1/_4$ in. drop across the width of the bearing seat is sufficient.

Girder Stop Bearings

For skewed structures with earth pressure against the end diaphragm (see Figure 9.3.2-4), the performance of girder stop bearings shall be investigated at Service Limit State. These bearings are placed vertically against the girder stop to transfer the skew component of the earth pressure to the abutment without restricting the movement of the superstructure in the direction parallel to centerline. In some cases bearing assemblies containing sliding surfaces may be necessary to accommodate large superstructure movements.

Girder Stop Design

Some type of transverse girder stop is required for all abutments in order to transfer earthquake load from the superstructure to the abutment. The girder stop shall be designed at the Extreme Limit State for the earthquake loading, any transverse earth pressure from skewed abutments, etc. Girder stops are designed using shear friction theory. The possibility of torsion combined with horizontal shear when the load does not pass through the centroid of the girder stop shall also be investigated.

Girder Stop Detail

The detail shown in Figure 7-38 may be used for bridges with no skew. Prestressed girders should be placed in final position before girder stops are cast to eliminate alignment conflicts between prestressed girders and girder stops. All girder stops should provide $^1/_8$ in clearance between the prestressed girder flange and the girder stop.

Note:

1. Girder Stops shall be constructed after placement of prestressed girders.
2. Elastomeric pads between girder and girder stops shall be placed after constructing the girder stops. The pads shall be cemented to girder stops with approved rubber cement.

Girder Stop Details
Figure 7-38

7.5.6 Abutment Expansion Joints

For structures without expansion joints, the earth pressure against the end diaphragm is transmitted through the superstructure. The compressibility of the expansion joint shall be considered in the design of the abutment for earthquake, temperature, and shrinkage when these forces increase the design load.

7.5.7 Open Joint Details

Vertical expansion joints extending from the top of footings to the top of the abutment are usually required between abutments and adjacent retaining walls to handle anticipated movements. The expansion joint is normally filled with premolded joint filler which is not water tight. There may be circumstances when this joint must be water tight; $^1/_8$ butyl rubber may be used to cover the joint. The open joint in the barrier should contain a compression seal to create a watertight joint. Figure 7-39 shows typical details that may be used. Aesthetic considerations may require that vertical expansion joints between abutments and retaining walls be omitted. This is generally possible if the retaining wall is less than 60 feet long.

Open Joint Details between Abutment and Retaining Walls
Figure 7-39

The footing beneath the joint may be monolithic or cast with a construction joint. In addition, dowel bars may be located across the footing joint parallel to the wall elements to guard against differential settlement or deflection.

On abutments with the end diaphragm cast on the superstructure, the open joints must be protected from the fill spilling through the joint. Normally, 1/8 in. butyl rubber is used to seal the openings. Figure 7.5.7-2 and Figure 7-40 show typical details using butyl rubber. Figure 7.5.7-2 is a detail for the horizontal joint between the abutment and the bottom of the end girder diaphragm. Figure 7-40 is a detail for the vertical joints between the wing wall and the end girder diaphragm. Other methods to protect the open joints must be well detailed in the plans. The Special Provision and Estimates unit can advise as to what types of joint materials would or would not require special provisions.

End Diaphragm on Girder
Figure 7-40

7.5.8 Construction Joints

To simplify construction, vertical construction joints are often necessary, particularly between the abutment and adjacent wing walls. Construction joints should also be provided between the footing and the stem of the wall. Shear keys shall be provided at construction joints between the footing and the stem, at vertical construction joints or at any construction joint that requires shear transfer. The Standard Specifications cover the size and placement of shear keys. The location of such joints shall be detailed on the plans. Construction joints with roughened surface can be used at locations (except where needed for shear transfer) to simplify construction. These should be shown on the plans and labeled "Construction Joint With Roughened Surface." When construction joints are located in the middle of the abutment wall, a pour strip should be used for a clean joint between pours. Details of the pour strip should be shown in the plans.

7.5.9 Abutment Wall Design

When the primary structural action is parallel to the superstructure or normal to the abutment face, the wall shall be treated as a column subjected to combined axial load and bending moment. Compressive reinforcement need not be included in the design of cantilever walls, but the possibility of bending moment in the direction of the span as well as towards the backfill shall be considered. A portion of the vertical bars may be cut off where they are no longer needed for stress.

Shrinkage and Temperature Reinforcement

The AASHTO requires a minimum temperature and shrinkage steel of 0.125 sq. in. per foot of wall. This is not sufficient to limit shrinkage cracks in thick walls. A more appropriate minimum temperature and shrinkage steel is taken from the ACI-83, minimum area of reinforcing steel per foot of the wall, in both directions on each face of the wall, shall be 0.011 times the thickness of the wall (in inches), spaced at 12 inches. On abutments that are longer than 60 feet, consideration should be given to have vertical construction joints to minimize shrinkage cracks.

The minimum cross tie reinforcement in the abutment wall is as follows. #4 tie bars with 180 degree hooks, spaced at approximately 2 feet center to center vertically and at approximately 4 feet center to center horizontally shall be furnished throughout the abutment stem in all but stub abutments, see Figure 7-41.

Cross Tie Details

Figure 7-41

7.5.10 Drainage and Backfilling

Three-inch (3 in.) weep holes shall be provided in all bridge abutment walls. These shall be located 6 inches above the final ground line at about 12 feet on centers. In cases where the vertical distance between the top of the footing and the bearing seat is greater than 10 feet, additional weep holes shall be provided 6 inches above the top of the footing. No weep holes are necessary in cantilever wing walls where a wall footing is not used.

The details for gravel backfill for walls, underdrain pipe and backfill for drains shall be indicated on the plans. The gravel backfill for walls shall be provided behind all bridge abutments. The underdrain pipe and gravel backfill for drains shall be provided behind all bridge abutments except abutments on fills with a stem wall height of 5 feet or less. When retaining walls with footings are attached to the abutment, a blockout may be required for the underdrain pipe outfall. Cooperation between Bridge and the district as to the drainage requirements is needed to guarantee proper blockout locations.

Underdrain pipe and gravel backfill for drains are not necessary behind cantilever wing walls. 3 foot thickness of gravel backfill for walls behind the cantilever wing walls shall be shown in the plans.

The backfill for walls, underdrain pipe and gravel backfill for drains are not included in bridge quantities, the size of the underdrain pipe should not be shown on the plans. Figure 7-42 illustrates backfill details.

Drainage and Backfill Details
Figure 7-42

7.6 Wing/Curtain Wall at Abutments

Particular attention should be given to the horizontal reinforcing steel required at fixed corners between abutment and wing/curtain walls. Since wall deflections are zero near the abutment, curtain walls and cantilever wing walls shall assume an At-Rest soil pressure. This increased loading can normally be reduced to an Active soil pressure at a distance (from the corner), equal to the average height of the wall under design. At this distance, the wall deflections are assumed large enough to allow the active state soil pressures to be developed. For the typical abutment, wing wall moments may be assumed to distribute stress to the outer 10 foot portion of the abutment wall. See GDM Section 15.4.2.7, “Active, Passive, and At-Rest Pressures”.

7.6.1 Traffic Barrier Loads

Traffic barriers should be rigidly attached to an approach slab that is cantilevered over the top of a wing/curtain wall or Structural Earth wall. The barrier collision load is applied directly to the approach slab. The yield line theory as specified in AASHTO LRFD Specifications article A13.3 is primarily for traffic barrier on bridge deck slabs and may not be applicable to traffic barrier on less rigid supports, such as retaining walls.

7.6.2 Wingwall BDM Design

The following wingwall design items should be addressed in the Plans.

A. For Strength Design of wing walls, vertical loads and moments may be distributed over 10 feet of the abutment wall and footing.

B. Footing thickness shall be not less than 1 foot 6 inches.

C. Exterior girder top flanges should be located (at the least) inside the curb line at the end pier.

D. For skewed bridges, modify the details on the traffic barrier and approach slab sheet so the expansion joint detailing agree. List appropriate manufacturers and model numbers for the expansion joint system. Generally, a 1 inch expansion joint with a 1inch open joint in the barrier is shown in the Plans, unless the bridge expansion joint design dictates otherwise.

7.6.3 Wing wall Detailing

All wing wall reinforcement should be a vertical grid and not follow a tapered bottom of wall. This allows for the steel to be placed in two layers that fits better with abutment reinforcing. Existing MicroGDS wing wall sheets conform to the LFD specifications. For consistency in design with the other bridge components, these wing walls sheets must be re-designed in accordance with the requirements of the AASHTO LRFD Specifications.

7.7 Footing Design

7.7.1 General Footing Criteria

The provisions given in this section pertain to both spread footings and pile supported footings.

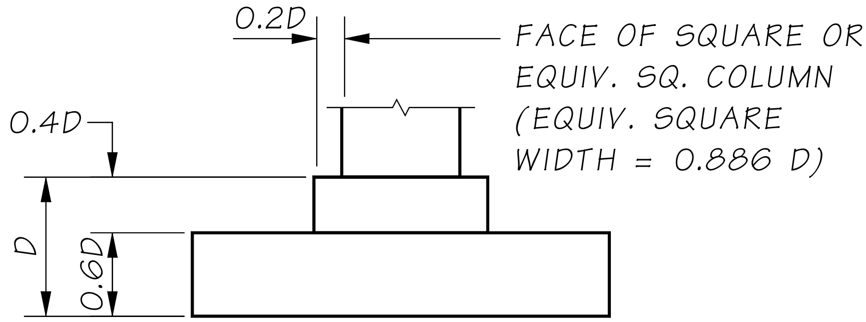
Minimum Cover and Footing Depth

Footings shall normally be rectangular in plan for both square and skewed bridges. The Geotechnical Report may specify a minimum footing depth in order to assure adequate bearing pressure. Stream crossings may require additional cover depth as protection against scour. The Hydraulic Section should be consulted on this matter. Footings set too low result in large increases in cost. The end slope on the bridge approach fill is usually set at the preliminary plan stage but affects the depth of footings placed in the fill. Figure 7-43 illustrates footing criteria when setting footing elevations.

Guidelines for Footing Cover and Depth
Figure 7-43

Pedestals

A pedestal is sometimes used as an extension of the footing in order to provide additional depth for shear near the column. Its purpose is to provide adequate structural depth while saving concrete. For proportions of pedestals, see Figure 7-44. Since additional forming is required to construct pedestals, careful thought must be given to the trade off between the cost of the extra forming involved and the cost of additional footing concrete. Also, additional foundation depth may be needed for footing cover. Whenever a pedestal is used, the plans shall note that a construction joint will be permitted between the pedestal and the footing. This construction joint should be indicated as a construction joint with roughened surface.



Pedestal Dimensions
Figure 7-44

7.7.2 Loads and Load Factors

The following Table 7-45 is a general application of minimum and maximum load factors as they apply to a generic footing design. Footing design must select the maximum or minimum Load Factors for various modes of failure for the Strength and Extreme Event Limit States.

The dead load includes the load due to structural components and non-structural attachments (DC), and the dead load of wearing surfaces and utilities (DW). The live load (LL) does not include vehicular dynamic load allowance (IM).

Designers are to note, if column design uses magnified moments, then footing design must use magnified column moments.

Sliding and Overturning, e_o	**Bearing Stress (e_c, s_v)**
LL_{min} = 0	LL_{max}
DC_{max}, DW_{max} for causing forces, DC_{min}, DW_{min} for resisting forces	DC_{max}, DW_{max} for causing forces, DC_{min}, DW_{min} for resisting forces
EV_{min}	EV_{max}
EH_{max}	EH_{max}
LS	LS

Load Factors
Table 7-45

7.7.3 *Geotechnical Report Summary*

The Geotechnical Branch will evaluate overall bridge site stability. Slope stability normally applies to steep embankments at the abutment. If stability is in question, a maximum service limit state load will be specified in the report. Bridge design will determine the maximum total service load applied to the embankment. The total load must be less than the load specified in the Geotechnical Report.

Based on the foundations required in the Preliminary Plan and structural information available at this stage, the Report provides the following geotechnical engineering results. For all design limit states, the total factored footing load must be less than factored resistance.

Plan Detailing

The Bridge Plans shall include the nominal bearing resistance in the General Notes as shown in Figure 7-46. This information is included in the Plans for future reference by the Bridge Office.

Pier	Strength and Extreme (q_n)	Service (q_{serv})
1	== KSF	== KSF
2	== KSF	== KSF

Figure 7-46

Bearing Capacity - Service, Strength and Extreme Limit States

The unfactored bearing capacity (q_n) may be increased or reduced based on previous experience for the given soils. The Geotechnical Report will contain the following information:

- Unfactored bearing capacity (q_n) for anticipated effective footing widths, which is the same for the strength and extreme event limit states
- Resistance factor for strength limit state (ϕ_b).
- Resistance factor for the extreme event limit state (ϕ_b) is 1.0
- Service bearing capacity (q_{ser}) and amount of assumed settlement
- Embedment depth requirements or footing elevations to obtain the recommended q_n

Sliding Capacity - Strength and Extreme Limit States

The Geotechnical Report will contain the following information to determine earth loads and the factored sliding resistance (Q_R). $Q_R = \phi \quad Q_n = \phi$

- Resistance factor for strength limit state (ϕ_τ)
- Soil parameters ϕ_{soil}, K_a, and γ for calculating Q_τ and active force (EH) behind abutment footings
- If passive earth pressure (Q_{ep}) is allowed at a footing,

  Soil parameters of ϕ_{soil}, K_p, γ and depth of soil in front of footing

  Resistance factor ϕ_{ep} for strength

Foundation Springs - Extreme Limit State

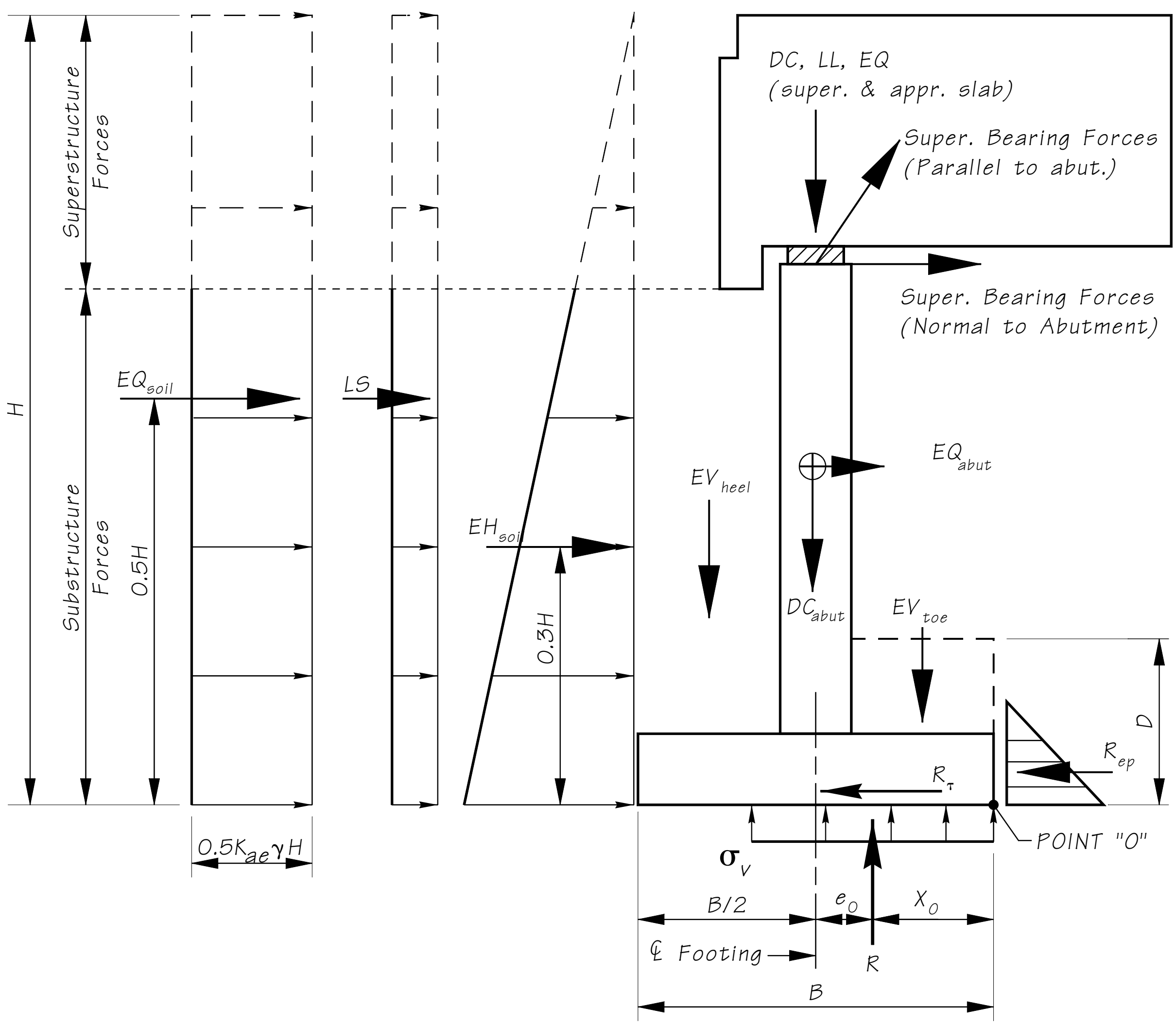
When a structural evaluation of soil response is required for a bridge analysis, the Geotechnical Branch will determine foundation soil/rock shear modulus and Poissons ratio (G and μ). These values will typically be determined for shear strain levels of 2% to 0.2%, which are typical strain levels for large magnitude earthquakes.

7.7.4 Spread Footing Structural Design

The following BDM Section is oriented towards abutment spread footing design. Spread footing designs for intermediate piers or other applications use the same concepts with the appropriate structural analysis. Structural designers should complete all design checks before consulting a problem with the geotechnical engineer. There may be several problem criteria that should be addressed in the solution.

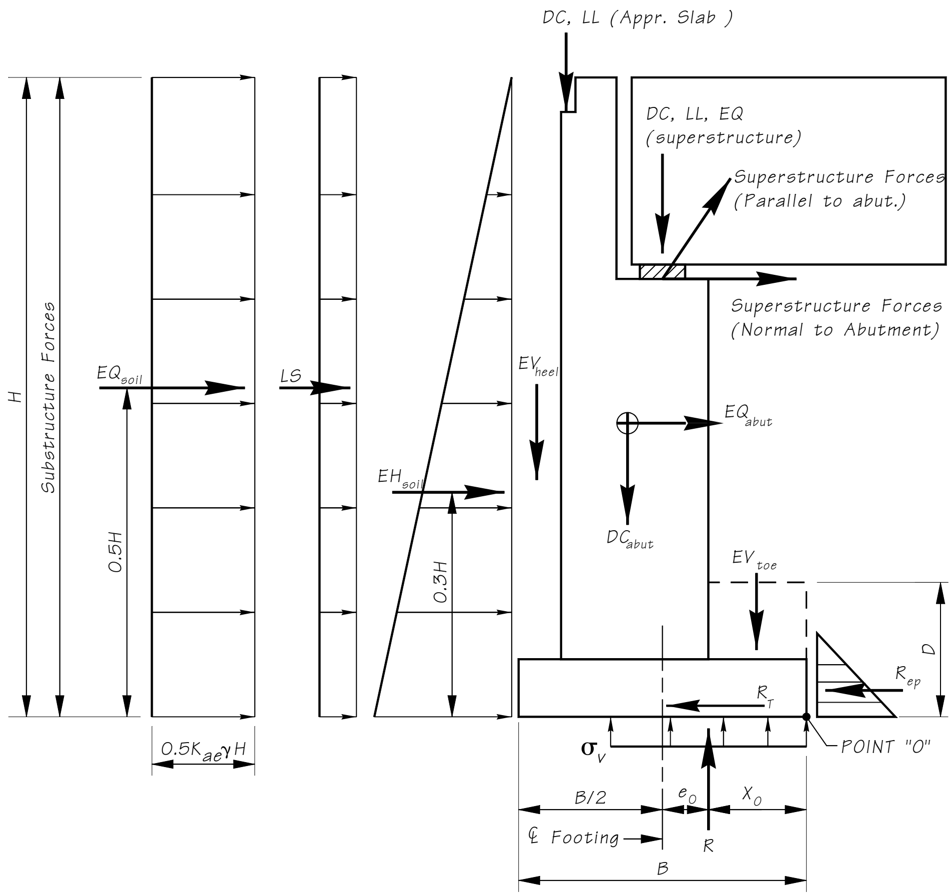
7.7.4.A Abutment Spread Footing Force Diagram

Figures 7-47 and 7-48 diagram the forces that act on abutment footings. Each limit state design check will require calculation of a reaction (R) and the location (X_o) or eccentricity (e_o). The ultimate soil passive resistance (Q_{ep}) at the toe is determined by the geotechnical engineer and is project specific.



Cantilever (End Diaphragm) Abutment Force Diagram

Figure 7-47



L-Abutment Force Diagram
Figure 7-48

7.7.4.B Bearing Stress

For geotechnical and structural footings design, the bearing stress calculation assumes a uniform bearing pressure distribution. For footing designs on rock, the bearing stress is based on a triangular or trapezoidal bearing pressure distribution. The procedure to calculate bearing stress is summarized in the following outline. See Abutment Spread Footing Force Diagrams for typical loads and eccentricity.

Step 1: Calculate the Resultant force (R_{str}), location (Xo_{str}) and eccentricity for Strength (e_{str}).

Xo_{str} = (factored moments about the footing base)/(factored vertical loads)

Step 2A: For Footings on Soil:

Calculate the maximum soil stress (σ_{str}) based on a uniform pressure distribution. Note that this calculation method applies in both directions for biaxially loaded footings. See AASHTO 10.6.3.1.5 for guidance on biaxial loading. The maximum footing pressure on soil with a uniform distribution is:

σ_{str} = R/B′ = R/2Xo = R/(B-2e), where B′ is the effective footing width.

Step 2B: For Footings on Rock:

If the reaction is outside the middle 1/3 of the base, use a triangular distribution.

σ_{str} max = 2R/3 Xo, where “R” is the factored limit state Reaction.

If the reaction is within the middle 1/3 of the base, use a trapezoidal distribution.

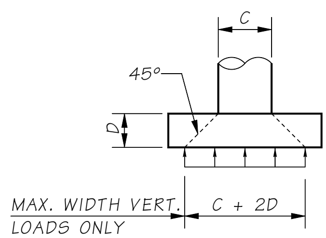
σ_{str} max = R/B (1+ 6 e / B^2)

In addition, WSDOT limits the maximum stress (P/A) applied to rock due to vertical loads only. This is because the rock stiffness approaches infinity relative to the footing concrete. The maximum width of uniform stress is limited to C+2D as shown in Figure 7-49.

Step 3: Compare the factored bearing stress (σ_{str}) to the factored bearing capacity ($\phi_{bc}q_n$) of the soil or rock. The factored bearing stress must be less than or equal to the factored bearing capacity.

$\sigma_{str} \leq \phi_{bc}q_n$

Step 4: Repeat steps 1 thru 3 for the Extreme Event limit state. Calculate Xo_{ext}, e_{ext}, and σ_{ext} using Extreme factors and compare the factored stress to the factored bearing ($\phi_b q_n$).



Footings on Rock
Figure 7-49

7.7.4.C Failure By Sliding

The factored sliding resistance (Q_R) is comprised of a frictional component (ϕ_τ Q_τ) and the Geotechnical Branch may allow a passive earth pressure component (ϕ_{ep} Q_{ep}). The Structural Engineer will calculate Q_R based on the soil properties specified in the Geotechnical Report. The frictional component acts along the base of the footing, and the passive component acts on the vertical face of a buried footing element. The factored sliding resistance should be greater than or equal to the factored horizontal applied loads.

$Q_R = \phi_\tau Q_\tau + \phi_{ep} Q_{ep}$

The Strength Limit State ϕ_τ and ϕ_{ep} are provided in the Geotechnical Report or AASHTO 10.5.5-1. The Extreme Event Limit State ϕ_τ and ϕ_{ep} are generally equal to 1.0.

Q_τ = (R) tan δ

δ = friction angle between the footing base and the soil

δ = tan ϕ for cast-in-place concrete against soil

δ = (0.8)tan ϕ for precast concrete

R = Minimum Strength and Extreme factors are used to calculate R

ϕ = angle of internal friction for soil

7.7.4.D Overturning Stability

Calculate the locations of the overturning reaction (R) for strength and extreme limit states. Minimum load factors are applied to forces and moments resisting overturning. Maximum load factors are applied to forces and moments causing overturning. Note that for footings subjected to biaxial loading, the following eccentricity requirements apply in both directions.

Strength limit state requires R to be located in the middle $^1/_2$ of the footing plan dimensions for soil and the middle $^3/_4$ of the footing dimensions for rock. See AASHTO 10.6.3.1.5.

Extreme event II limit state (EQ load factor = 0) requires resultant force to be located in the middle $^2/_3$ of the footing plan dimensions for soil and rock.

7.7.4.E Footing Settlement

The service limit state bearing capacity (q_{ser}) will be a settlement-limited value, typically 1 inch.

Bearing Stress = $\sigma_{ser} < \phi q_{ser}$ = Factored nominal bearing

Where, q_{ser} is the unfactored service limit state bearing capacity and ϕ is the service resistance factor. In general, the resistance factor (ϕ) will be equal to 1.0.

For immediate settlement (not time dependent), both permanent dead load and live load should be considered for sizing footings for the service limit state. For long-term settlement (on clays), only the permanent dead loads should be considered.

If the structural analysis yields a bearing stress (σ_{ser}) greater than the bearing capacity, then the footing must be re-evaluated. The first step would be to increase the footing size to meet bearing capacity. If this leads to a solution, recheck layout criteria and inform the geotechnical engineer the footing size has increased. If the footing size cannot be increased, consult the geotechnical engineer for other solutions.

7.7.4.F Concrete Design

Footing design will be in accordance with AASHTO Section 5.13.3 for footings and the general concrete design of AASHTO Chapter 5. The following Figure 7-50 illustrates the modes of failure checked in the footing concrete design.

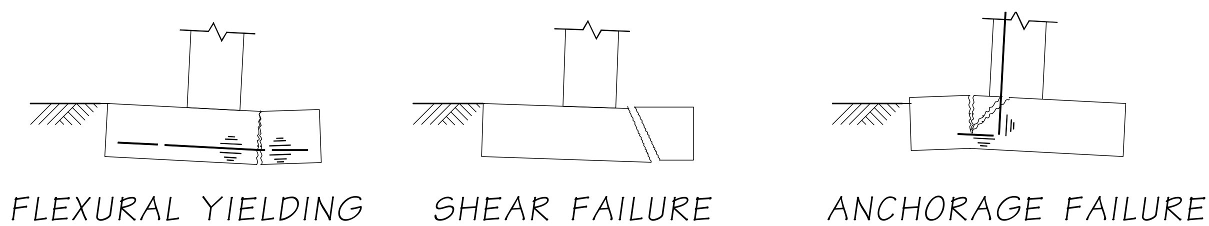


Figure 7-50

Footing Thickness and Shear

The minimum footing thickness shall be 1 foot 6 inches. The minimum plan dimension shall be 4 feet 0 inches. Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements (with or without reinforcement). If concrete shear governs the thickness, it is the Engineer's judgment, based on economics, as to whether to use a thick footing unreinforced for shear or a thinner footing with shear reinforcement. Generally, shear reinforcement should be avoided but not at excessive cost in concrete, excavation, and shoring requirements. Where stirrups are required, place the first stirrup at d/2 from the face of the column or pedestal. For large footings, consider discontinuing the stirrups at the point where $v_u = v_c$.

Footing Force Distribution

The maximum shear stress in the footing concrete shall be determined based on a triangular or trapezoidal bearing pressure distribution, see AASHTO 5.13.3.6. This is the same pressure distribution as for footing on rock, see BDM Section 7.7.4B.

Vertical Reinforcement (Column or Wall)

Vertical reinforcement shall be developed into the footing to adequately transfer loads to the footing. Vertical rebar will be bent 90º and extend to the top of the bottom mat of footing reinforcement. This facilitates placement and minimizes footing thickness. Bars in tension shall be developed using 1.25 Ld. Bars in compression shall develop a length of 1.25 Ld, prior to the bend. Where bars are not fully stressed, lengths may be reduced in proportion, but shall not be less than 3/4 Ld.

The concrete strength used to compute development length of the bar in the footing shall be the strength of the concrete in the footing. The concrete strength to be used to compute the section strength at the interface between footing and a column concrete shall be that of the column concrete. This is allowed because of the confinement effect of the wider footing.

Bottom Reinforcement

Concrete design will be in accordance with AASHTO. Reinforcement shall not be less than #6 bars at 12-inch centers to account for uneven soil conditions and shrinkage stresses.

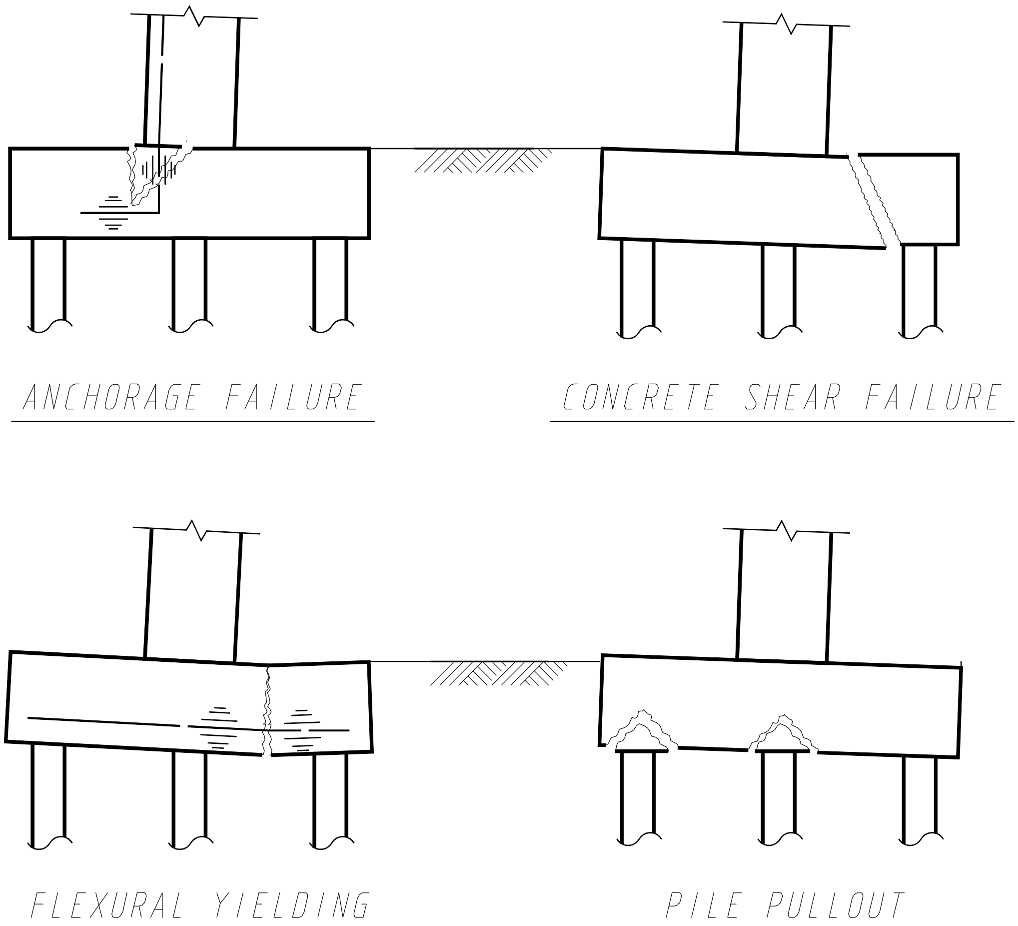
Top Reinforcement

Top reinforcement shall be used in any case where tension forces in the top of the footing are developed. Where columns and bearing walls are connected to the superstructure, sufficient reinforcement shall be provided in the tops of footings to carry the weight of the footing and overburden assuming zero pressure under the footing. This is the uplift earthquake condition described under "Superstructure Loads." This assumes that the strength of the connection to the superstructure will carry such load. Where the connection to the superstructure will not support the weight of the substructure and overburden, the strength of the connection may be used as the limiting value for determining top reinforcement. For these conditions, the AASHTO requirement for minimum percentage of reinforcement will be waived. Regardless of whether or not the columns and bearing walls are connected to the superstructure, a mat of reinforcement shall normally be provided at the tops of footings. On short stub abutment walls (4 feet from girder seat to top of footing), these bars may be omitted. In this case, any tension at the top of the footing, due to the weight of the small overburden, must be taken by the concrete in tension.

Top reinforcement for column or bearing wall footings designed for two-way action shall not be less than #6 bars at 12-inch centers, in each direction while top reinforcement for bearing wall footings designed for one-way action shall not be less than #5 bars at 12-inch centers in each direction.

7.7.5 Footing Concrete Design on Pile Supports

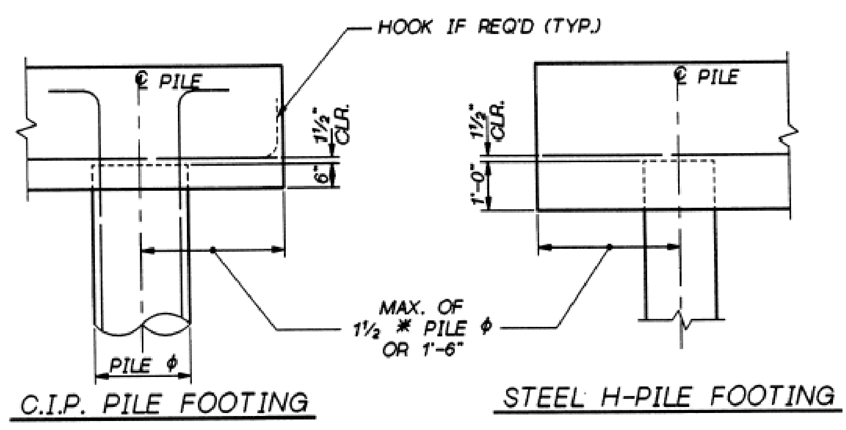
The minimum footing thickness shall be 2 feet 0 inches. The minimum plan dimension shall be 4 feet 0 inches. Distance from center of pile to footing edge for all pile types shall be a maximum of 1.5 times the pile diameter or 1 foot 6 inches. Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements. The use of strut and tie modeling is recommended for the design of all pile caps and pile footings. Figure 7-51 identifies the modes of failure that should be investigated for general pile cap/footing design.



Pile Footing Modes of Failure
Figure 7-51

7.7.5.A Pile Embedment, Clearance, and Rebar Mat Location

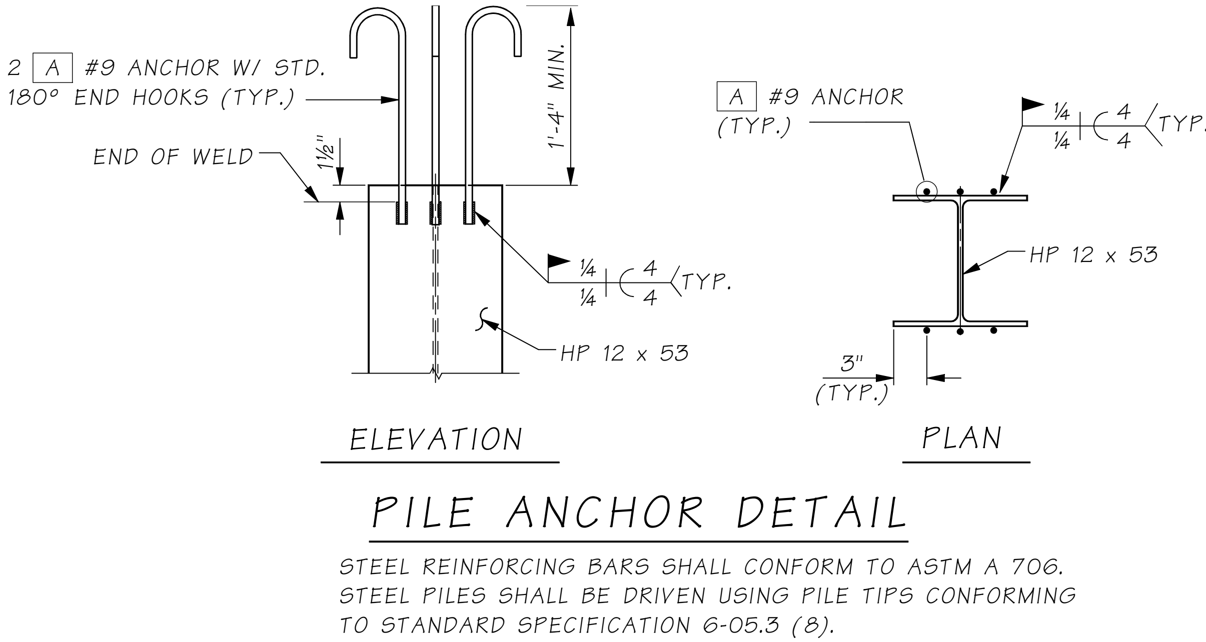
All piles shall have an embedment in the concrete sufficient to resist moment, shear, and axial loads. Steel H-Pile or timber piles are embedded a minimum of 12 inches into the concrete where a moment or tension connection is not required. Cast-in-place concrete piles with reinforcing extending into footings are embedded a minimum of 6 inches. The clearance is 1½ inches between the bottom mat of footing reinforcement and the top of pile. See Figure 7.7.5.1.1 for the minimum pile clearance to the edge of footing.



Pile Embedment and Reinforcing Placement
Figure 7-52

7.7.5.B Pile Pullout and Details

Piles subject to uplifting forces must be adequately detailed to connect the footing to the pile. Figure 7.7.5.1.2 details extended reinforcement welded to the pile. Skin friction due to the bond between pile and footing is not allowed in the design of the connection.



Sample Steel Pile Anchorage
Figure 7-53

7.7.5.C Concrete Design

In determining the proportion of pile load to be used for calculation of shear stress on the footing, any pile with its center 6 inches or more outside the critical section shall be taken as fully acting on that section. Any pile with its center 6 inches or more inside the critical section shall be taken as not acting for that section. For locations in between, the pile load acting shall be proportioned between these two extremes. For calculation of moment on the footing, any pile with its center outside of the section shall be taken at full load. Any pile with its center inside of the section shall not be assumed to contribute to that amount.

7.8 Drilled Shafts

7.8.1 Drilled Shaft Design

The factored axial resistance of the drilled shaft (R) is generally composed of two parts: the nominal top resistance (R_p) and the nominal side friction resistance (R_s). The general formula is as follows, where φ is the limit state resistance factor.

$$R = \varphi_p R_p + \varphi_s R_s$$

The total factored shaft loading must be less than the factored axial resistance. R_p and R_s are treated as independent quantities although research has shown that the tip (or bearing) resistance and side resistance have some independence. The factored nominal tip and side resistance will be stated in the Geotechnical Report for the bridge.

Plan Detailing

The Bridge Plans shall include the nominal side and tip resistance for the service limit state in the General Notes, as show in Figure 7-54. This information is included in the Plans for reference by the Contractor. If the shaft has significant bearing resistance, the contractor shall take extra care to clean the bottom of the shaft before placing the shaft concrete.

Pier	Nominal Side Resistance (R_s)	Nominal Tip Resistance (R_p)
1	== KIPS	== KIPS
2	== KIPS	== KIPS

Figure 7-54

A. **Reinforcing and Concrete Strength**

Due to soil conditions and construction methods, concrete may not be placed in the dry. A reduction in concrete strength used for design shall be as follows:

1. Shaft diameter 4 feet 0 inches or less – assumed concrete compressive strength shall be 0.85 fc′. Concrete placed by tremie method is confined to small area and segregation is reduced. Cover requirement – 3-inch minimum to 6-inch maximum.
2. For shaft diameters of 4 feet 6 inches or more, use 0.6fc′. Cover requirement are 6-inch minimum to 12-inch maximum.
3. Reinforcing shall be detailed to minimize congestion. Longitudinal reinforcing extending into footing should be straight. If hooked, detail so that casing can be removed while placing concrete. Percentage of reinforcing shall be 0.5 percent minimum and 4 percent maximum. Use of two concentric circular cages shall be avoided.

B. **Design of Drilled Shafts**

The analysis will provide the design loading for the shaft concrete and reinforcement design. The following guidelines apply.

1. Column Assumptions: Normally, the soil surrounding a foundation element provides bracing against a buckling failure. For this reason, the drilled shaft can be designed as a short column when the shaft is entirely below the groundline. When the shaft extends above the ground a check for slenderness may apply. The effects of scour shall be considered in the analysis.

2. Axial Load, Bending Moment, and Shear: The axial load along the shaft varies due to the side friction. It is considered conservative, however, to design the shaft for the full axial load plus the maximum moment. The entire shaft normally is then reinforced for this axial load and moment. Longitudinal reinforcing should not be less than 0.5 percent of the area of concrete. Design shaft for axial load bending movement and shear as a column.

7.8.2 Lateral Load Analysis

Shafts must be analyzed in accordance with BDM Section 7.2, Foundation Modeling. In general, FEM bridge model analysis iteratively matches deflections and load between a non-linear soil/structure interaction analysis.

In general, only Service Limit and EQ loading are used for the seismic bridge analysis to establish load distribution in the structure. The results then are factored for the design of the bridge components. Note, if Strength Limit state controls the design, then EQ loading would not apply.

In some cases, the depth required for shaft stability based on a lateral analysis may control the depth of shaft, rather than bearing capacity or uplift. Cases such as bridge sites with soft or liquefiable soils. Shaft or pile stability will require engineering judgment and experience.

A. For seismically controlled designs, the bigger (stiffer) the shaft the more movement can be tolerated at the shaft tip. A seismic analysis will predict the maximum deflections and stresses and the engineer must determine a safe shaft depth to survive the event. Small pile fixity generally refers to the point of fixity for column design, or the first inflection point when observing deflection. The pile tip for small shafts/piles, one to two feet in diameter, should be determined at the location of approximately the second point of inflection. An acceptable movement at the tip during an Extreme Event has yet to be determined. In general, the smaller the better. Since these shafts/piles are relatively flexible in the soil, it is possible to have pile tips at the 2nd point of inflection with little or no movement (drift) and not have deep tip elevations that are costly.

B. Medium sized shafts, three to eight feet in diameter, tipping the shafts should consider an elevation near the midpoint of the 1st and 2nd inflection points. An acceptable movement at the tip during an Extreme Event has yet to be determined. In general, past practice has been the smaller the better based on the experience of small flexible piles.

C. Large shafts, greater than 10-foot diameter, will transfer significantly more stresses to the soil and much deeper in the soil than flexible piles. Tipping for large shafts should consider an elevation between the midpoint and near the quarter point of the 1st and 2nd inflections. An acceptable movement at the tip during an Extreme Event has yet to be determined.

The static parameters represent the soil behavior for short-term transient loads such as wind, ice, temperature, and vessel impact. For earthquake loads, the seismic and static soil properties will be the same if the soils present have a stiffness which does not degrade with time during shaking.

If liquefiable soils are present, both static and liquefied soil properties are provided in the Geotechnical Report. Often, the highest acceleration the bridge sees is in the first cycles of the earthquake, and liquefaction tends to occur toward the middle or end of the earthquake. Therefore, early in the earthquake, loads are high, soil-structure stiffness is high, and deflections are low. Later in the earthquake, the soil-structure stiffness is lower and deflections higher.

If liquefaction can occur, the bridge should be analyzed twice. The first analysis uses the static soil conditions, which yields higher moment and shear to design the shaft (and column). The second analysis uses the liquefied soils to evaluate the bridge Extreme Event deflections. The intent here is to bracket the structure response. The designer will have to determine the acceptable maximum lateral deflection.

7.9 Piles and Piling

Plan Pile Resistance

The Bridge Plan General Notes shall denote pile size and Nominal Driving Resistance (or Ultimate Bearing Capacity) in Tons. This information is used by the contractor to determine the pile thickness and size the hammer to drive the piles. The resistance for several piers may be presented a table as shown in Figure 7-55. The Nominal Driving Resistance (R_{ndr}), as stated in the Geotechnical Report, will be shown in the Plans as "Ultimate Bearing Capacity". See WSDOT GDM, Section 8.12 for Geotechnical Design of the pile resistance. If overdriving the piles is required to reach the minimum tip elevation, see BDM 9.7.6, the estimated amount of overdriving (Tons) will be described in the Bridge Special Provisions.

PIER	ULTIMATE BEARING CAPACITY (Nominal Driving Resistance, R_{ndr})
1	== Tons
2	== Tons

Figure 7-55

The total factored pile axial loading must be less than ϕR_n for the pile design. Designers should note that the driving resistance might be greater than the design loading for liquefied soil conditions. This is not an overdriving condition. This is due to the resistance liquefied soils being ignored for design, but included in the driving criteria to place the piles.

7.9.1 Pile Types

This section of the BDM describes the piling used by the Bridge Office and their applications. Steel sheet piles are normally used for cofferdam and shoring and cribbing, but are usually not made a part of permanent construction. Piles should not be used where spread footings can be used at allowable basic bearing pressures of approximately 2 to 3 ton/sq. ft. or greater. Where heavy scour conditions may occur, pile foundations should be considered in lieu of spread footings. Where large amounts of excavation may be necessary to place a spread footing, pile support may be more economical.

A. Precast and Prestressed Concrete and Cast-In-Place Concrete Piles

Precast and prestressed concrete piles are no longer used for WSDOT bridge design. However, Standard Plans are available for prestressed piles. Precast Prestressed Concrete piles are hexagonal, square, or circular in cross-section and are prestressed to allow longer handling lengths. Again, close length determination at time of driving the test pile is important. Precast prestressed concrete piles are usually specified in accordance with Standard Plans such as Sheet E-4 for 13-inch diameter piles and Sheet E-4a for 16- and 18-inch diameter piles.

Cast-in-Place Concrete (CIP) Piles utilize driven steel pipe casings, which are then filled with concrete. The bottom of the casing is capped with a suitable flat plate before driving. The Geotechnical Division may specify special tips when difficult driving is expected.

Steel Pipe Casing

The Geotechnical Division will determine the minimum wall thickness based on driving conditions. The design for driving shall be based on a steel shell thickness of $^1/_4$ inch for piles less than 14 inches in diameter, $^3/_8$ inches for piles 14 to 18 inches in diameter, and $^1/_2$ inch for larger piles.

Steel casing for cast-in-place piling should be designated by nominal size, not inside diameter for 24 inch and smaller pile casings. Both ASTM A53 and A252 pipe is purchased by nominal size (outside diameter) and wall thickness, see specification for dimensions. These are the commonly supplied pipe specifications. A pipe thickness should not be stated in the plans. The Standard Specifications for pile driving states the contractor shall determine the pile casing thickness required for driving and increase minimum thickness specified as necessary."

CIP Pile Design

Class 4000 P Concrete shall be specified for inside the pile. The top 10 feet of concrete in the pile is to be vibrated.

For the pile stiffness and foundation modeling, the full cross section of the steel shell minus $^1/_{16}$ inch for corrosion will be used in determining the pile stiffness and foundation modeling. This thickness can also be considered as confinement reinforcement for the internal cage except at pile/footing interface. The moment of inertia of the pile is computed by adding the components

$$I\ pile_{conc} = I_{conc} + (n)(I_{shell}) + (n)(I_{reinf})$$

For pile structural design, the reinforcement alone shall be sufficient to resist the total moment throughout the length of pile without considering the shell. The minimum reinforcement shall be 0.5 percent of the gross concrete area for Seismic Performance Categories A and B, and 0.75 percent for Category C as required per AASHTO Seismic Guideline Specifications, Chapter 6. No less than four No. 5 bars shall be used. The reinforcement shall extend above the pile into the footing a distance equal to 1.25 1d (tension).

A steel reinforcing cage shall be used to tie the pile to the cap/footing. The vertical steel reinforcing bars, above the top of the pile, shall be tied together with closely spaced hoops or spirals as required by the seismic guide specifications. Inside the pile, No. 4 hoops at 12-inch centers is minimum required for Category B and 9-inch centers for Category C.

B. Steel H Piles

Steel piles are normally used where there are hard layers that must be penetrated in order to reach an adequate point bearing stratum. Steel stress is generally limited to 9.0 ksi (working stress) on the tip. H piling can act efficiently as friction piling due to its large surface area. Do not use steel H piling where the soil consists of only moderately dense material. In such conditions, it may be difficult to develop the friction capacity of the H piles and excessive pile length may result. The bridge layout will denote steel piles with capacity and size, e.g., steel pile 70-ton HP 12 x 53.

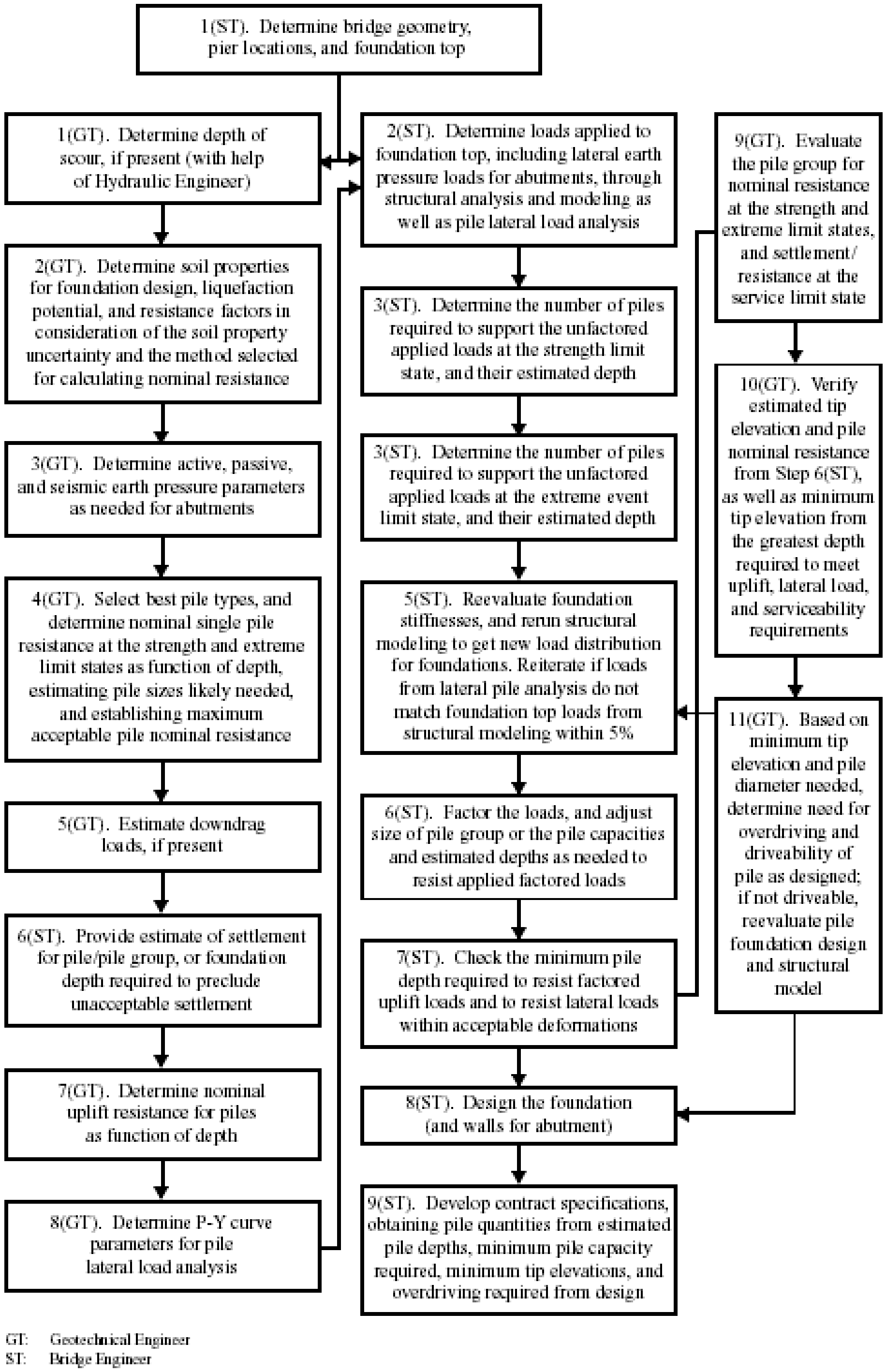
C. Timber Piles

Timber piles have the lowest cost per foot of any of the pile types. Timber piles may be untreated or treated. Untreated piles are used only for temporary applications or where the entire pile will be permanently below the water line. Composite piles, treated and untreated, may be used if the pile length is long and a splice will be required. Where composite piles are used, the splice must be located below the permanent water table. If doubt exists as to the location of the permanent water table, treated timber piles shall be used.

Where dense material exists, consideration should be given to allowing jetting (with loss of uplift capacity), use of shoes, or use of other pile types.

7.9.2 *Pile Design Flow Chart*

Figure 7-56 provides a flowchart which illustrates the design process and the interaction between the structural and geotechnical engineers needed for pile foundation design. **Once the pile analysis and design are completed in the Bridge and Structures Office, the Geotechnical Division is to be contacted for final review and comment.**



Design Flowchart for Pile Foundation Design
Figure 7-56

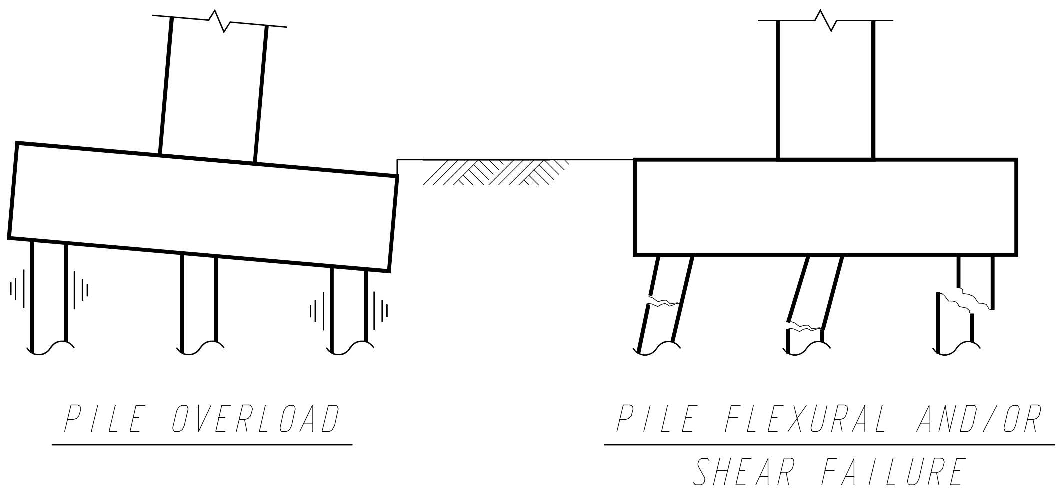
7.9.3 General Pile Design

The objective of pile structural design is to determine the pile group geometry, minimum tip elevations and estimated pile quantity. The bridge structural design may include characterization of the pile foundation for purposes of modeling the overall structure, especially for seismic design.

Pile Spacing

Generalized pile spacing is shown in Section 9.6 for each type of pile. Be aware that the action of the pile group for friction piles may be quite different than for point bearing piles, in that the group can fail as a unit at a lower load than the summation of the individual pile capacities. This effect is accounted for in Chapter 4, "Modeling Pile Foundation."

For point bearing piles, the spacing is a minimum of 3 feet, except for timber piles where the minimum spacing is 3 feet 3 inches. Where the load distribution of the pile is partially point bearing and partially friction, consider using an intermediate spacing value.



Footing Pile Modes of Failure

Figure 7-57

Column Action

Consideration shall be given to the pile acting as a column. Piles that support footings or pile caps that are not in contact with the soil below them must be treated as columns subject to bending and axial load. Piles that extend above the ground surface shall be analyzed by the appropriate column design procedures. Piles that are driven through very weak soils should be designed for reduced lateral support, using information from the Geotechnical Division as appropriate. AASHTO 10.7.4.2 may be used to estimate the column length for buckling. Piles driven through firm material normally can be considered fully supported for column action (buckling not critical) below the ground.

Friction vs. Point Bearing Piles

Pile axial resistance may be friction, point bearing, or a combination of both. In the absence of a soil layer that can offer adequate resistance to develop full point bearing, normally the pile shall be considered to be acting as a friction pile. The Geotechnical Report will describe the type soil resistance that supports the pile load. The soil conditions that support the pile may affect several structural properties, such as: rate of pile elastic shortening, effects of group action and hence spacing, pile column stability, and ability to resist lateral forces.

Pile Splices

Pile splices shall be avoided where possible. If splices may be required in timber piling, a splice shall be detailed on the plans. Splices between treated and untreated timber shall always be located below the permanent water line. Concrete pile splices shall have the same strength as unspliced piles.

Uplift Capacity

The ability of a pile to carry uplift loads is highly dependent upon the strata into which it is driven. When uplift is included in pile analysis, the following items apply:

1. The pile must be a friction pile and over 10 feet in length. Whenever uplift is to be used in the pile design, the Foundation Engineer shall be consulted. Uplift will not be used for pile design when piles are full point bearing piles.
2. In all cases of uplift, the connection between the pile and the footing must be carefully designed and detailed. The bond between the pile and the seal may be considered as contributing to the uplift resistance. This bond value shall be limited to 10 psi. The pile must be adequate to carry tension throughout its length. For example, a timber pile with a splice sleeve could not be used.
3. Preboring, jetting, or spudding must not be used to aid in driving the pile and must be so noted in the plans or special provisions.

7.9.4 Pile Axial Design

The Geotechnical Report will provide the nominal axial resistance (R_n) as either a dynamic or static resistance. The factored pile load ($P_{u\ pile}$) for Strength and Extreme limit states must be less than the limit state resistance φR_n specified in the Geotechnical Report.

Axial Analysis

Generally, pile groups are assumed to act as a rigid body with rotations about the centroidal axis for strength design. Factored loads are applied to the pile cap and the pile ($P_{u\ pile}$) is determined as follows:

$$(P_{u\ pile}) = (P_{u\ pile\ group})/N + M_{u\ group}\ C/I_{group} + \varphi DD$$

where,

$M_{U\ group}$	=	Factored moment applied to the pile group. This includes eccentric LL, DC, centrifugal force (CE), etc. Generally, the dynamic load allowance (IM) does not apply.
C	=	Distance from the centroid of the pile group to the center of the pile under consideration
I_{group}	=	Moment of inertia of the pile group
N	=	Number of piles in the pile group
$P_{U\ pile\ group}$	=	Factored axial load to the pile group
φDD	=	Factored Down Drag forces specified in Geotechnical Report

Determine the number of piles required in the pile group such that the factored load in any pile in the pile group is not greater than the factored resistance. The pile weight will be normally neglected but may be added to the foundation dead load.

Extreme Event Design

A foundation spring support is usually required to represent the pile group in a dynamic bridge model. See BDM Section 7.2, Foundation Modeling for a method to calculate the axial or other matrix spring coefficients.

Block Failure

For the strength and extreme event limit states, if the soil is characterized as cohesive, the pile group capacity should also be checked for the potential for a "block" failure, AASHTO 10.7.3.10 applies. Compare the factored loads for each limit state to the factored block resistance. If a block failure appears likely, increase the group size so that a block failure is prevented.

Pile Uplift

Piles may be designed for uplift if specified in the Geotechnical Report.

Downdrag force

Downdrag forces are treated as load to the pile when designing for axial capacity. Do not include downdrag forces in the structural analysis of the bridge. Geotechnical Design of Down Drag loads are described in WSDOT GDM, Section 8.6.2.

7.9.5 Pile Lateral Design

The strength limit state for lateral resistance is only structural, though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state and this limit state is reached, in the general case, when the nominal combined bending, shear, and axial resistance is reached.

Piles resist horizontal forces by a combination of internal strength and the passive pressure resistance of the surrounding soil. The capacity of the pile to carry horizontal loads should be investigated using a soil/structural analysis. For more information on modeling individual piles or pile groups, see BDM Section 7.2, Foundation Modeling.

Battered Piles

Battered piles are not recommended to resist seismic lateral loads. The lateral force that can be resisted by a single battered pile is limited by a function of applied vertical dead load, and this must not be exceeded. Maximum batter shall be $4^1/_2$:12. Piles with batters in excess of this become very difficult to drive and the bearing values become difficult to predict. Ensure that battered piling do not intersect piling from adjacent footings within the maximum length of the piles.

7.9.6 Pile Tip Elevations and Quantities

Pile length quantities provided to PS&E are based on the estimated tip elevation given in the Geotechnical Report or the depth required for design whichever is greater. If the estimated tip elevation given in the Geotechnical Report is greater than the design tip elevation, overdriving the pile will be required. The Geotechnical Engineer should be contacted to evaluate driving conditions. A special provision to the Standard Construction Specifications is normally required to alert the contractor of the additional costs to drive these piles.

Minimum pile tip elevations provided in the Geotechnical Report may need to be adjusted to lower elevations depending on the results of the lateral, axial, and uplift analysis. This would become the minimum pile tip elevation requirement for the contract specifications. If adjustment in the minimum tip elevations is necessary, or if the pile diameter needed is different than what was assumed for the Geotechnical Report, the Geotechnical Division MUST be informed so that pile drivability can be re-evaluated. See BDM Section 9.8 for more information on tipping piles.

Note that lateral loading and uplift requirements may influence (possibly increase) the number of piles required in the group if the capacity available at a reasonable minimum tip elevation is not adequate. This will depend on the soil conditions and the loading requirements. For example, if the upper soil is very soft or will liquefy, making the minimum tip elevation deeper is unlikely to improve the lateral response of the piles enough to be adequate. Adding more piles to the group or using a larger pile diameter to increase the pile stiffness may be the only solution.